HYDRAULIC MODEL STUDIES OF LITTLE RIVER GAGING STATION B

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CONTENTS

Illustrations

Tables

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SUMMARY

Weirs to control and measure low flows in Little River at Gaging Station B of the Southeast Watershed Research Center were designed and models tested at the Water Conservation Structures Laboratory, Stillwater, Okla. The weirs are placed immediately downstream of the two bridges that span the river at this location. Model studies were used to develop a stilling basin which would protect the weirs and the bridges from scour. The head-discharge relationships were also determined by model studies and full-scale tests for the lowest flows.

INTRODUCTION

Gaging Station B lies below the 126-square-mile drainage area of the Little River Watershed which extends approximately 20 mi through Tift, Turner, and Worth Counties in Georgia. It is one of 18 proposed gaging stations on the river and its tributaries, to be operated by the Southeast Watershed Research Center, Agricultural Research Service, U. S. Department of Agriculture, Athens, Ga. Discharge measurements at these gaging stations provide the streamflow data to characterize the hydrology of the Southern Coastal Plains.

The gaging station site consists of two highway bridges which control the large flow rates and thus provide a way for measuring these rates. Low flows, however, require a more sensitive control than the bridge openings can provide to measure flow rate accurately. Weirs placed just below the bridge openings were proposed to provide such control. There were serious questions about the effect of these weirs on the capacity and the stability of the bridges. Therefore, 1:30 scale model tests were made at the ARS Water Conservation Structures Laboratory in Stillwater to answer these questions and to develop a suitable design. The 1:30 scale of the complete installation was used to calibrate the gaging station. In addition to Station B model tests, full-scale tests were made to determine rating coefficients for V-notch and horizontal weirs of the crest shape used in these gaging stations. These tests, the results, and recommendations are presented in this report.

THE GAGING STATION SITE

Station B is located where Tift County Road No. S1981 crosses the main stream of Little River, approximately 4 mi west of Tifton, Ga. The streamflow is discharged beneath two concrete-girder bridges supported by steel piles. The west bridge opening on the main channel is skewed 50° 49′ with the centerline of the road and is 204.7 ft in length. The east relief bridge opening is perpendicular to the road and is 77 ft in length. Bents comprised of four 1-ft-diameter steel piles are used to support each 26- to 30-ft span. The channel and floodplain areas are covered with a heavy growth of trees and underbrush except for the road right-of-way, which is cleared on each side of the county road.

THE LOW-FLOW CONTROL

Weirs located downstream of the bridges were selected for the low-flow control. This placement of the control allows current meter measurements to be made from the bridges in an area of relatively undisturbed flow during high-flow events.

A 16-inch broad-crested V-notch weir 3 ft deep with 1-on-10 side slopes located in the center of the opening was chosen for the west bridge opening. This type of weir provides sensitivity at very low flows, accuracy over a broad

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range of flows, relative economy of installation, and freedom from maintenance. Each side of the V-notch weir is flanked by a horizontal weir having the same cross section as the V-notch and extending to the training wall. The total length of this weir is 230 ft including the V-notch. A horizontal weir, without a V-notch, was placed downstream of the east bridge opening. Its crest is 0.5 ft higher than the level of the horizontal portion of the west weir crest. The length of the east weir is 100 ft.

Flow distribution data from U.S. Geological Survey Station 3180, Little River, Adel, Ga., indicate that approximately 70% of the average annual flow occurs at rates less than 530 ft^3/s. This is the capacity of the V-notch weir at a 3-ft head, with free discharge. Thus, if free discharge can be provided for the flow through the V-notch weir, 70% of the average annual flow volume can be measured with an overall error of ±5%.[2]

The low gradient of Little River at Gaging Station B makes it difficult and impractical to provide an overfall of sufficient height to permit free discharge for the full range of discharge rates expected. The capability to measure 70% of the average annual flow without submergence seemed practical and was accepted as a basis for the design of the flow-measuring station. The lowest elevation above the stream bed at which the V-notch could be placed and still have free overfall when flowing at capacity needed to be determined. This required an estimate of the normal depth of channel flow at the gaging station site.

Estimates of normal flow depth are usually made by water surface profile computation. This involves applying the laws of hydraulics to the geometry of the channel. However, the river had no well-defined channel at the gaging station site; it spread over the marshy, brush- and tree-covered bottom land. So direct measurement was used to obtain the water surface elevation data. Temporary staff gages were placed upstream and downstream of the west bridge. Flows were measured and staff gage readings were made. The relationship between flow rate and water surface elevation was determined. From this information and from knowledge of the hydraulics of V-notch weirs, the minimum elevation for the V-notch was determined. In terms of the arbitrarily assigned elevation of 100 for the construction benchmark on the curb of the west bridge, the elevation of the V-notch is 88.0. The crests of the horizontal weirs at the west bridge then are at elevation 91.0. The crest of the horizontal weir at the east bridge is at elevation 91.5.

[2] Herschy, R. W. 1969. The evaluation of errors at flow measurement stations. Water Resources Board T. N. 11. Reading Bridge House, Reading, Berkshire, U. K. 31 pp.

THE MODEL

A 10- by 40-ft test basin at the Water Conservation Structures Laboratory was used for the model studies. The size of the basin limited the length scale to 1:30. Since the flow over a weir is controlled by the force of gravity on a free water body, similarity of model and prototype was based on the Froude model law. The model-prototype relationships are shown in table 1.

TABLE 1.—*1:30 model-prototype transfer relationships for Little River Gaging Station B*

Property	Model scale
Length	1:30
Area	1:900
Velocity and time	1:5.48
Discharge	1:4930

Description of 1:30 Model

The modeled area is shown in figure 1. It included the cleared right-of-way on each side of the road 90 ft upstream and 105 ft downstream of the road centerline, and 1,035 ft of roadway (including the two bridges). The scaled-down width of the approach to the model was 34.5 ft. Therefore, the model had to be oriented 90° to the normal direction of flow in the 10- by 40-ft basin. The model of the bridges was built of wood to scale from field measurements. Three 6-inch-wide concrete cutoff walls were used to prevent seepage. These are shown in figure 2. The bridges were supported on the center cutoff wall and the other two walls were used to seal the upstream and downstream edges of the model.

The topography was modeled from information furnished by the Southeast Watershed Research Center. The fixed field topography was set to grade using a 1- to 2-inch thickness of concrete on a compacted sand bed. The channel downstream of each weir was reserved for a movable bed. The completed model without the movable bed material is shown in figure 3.

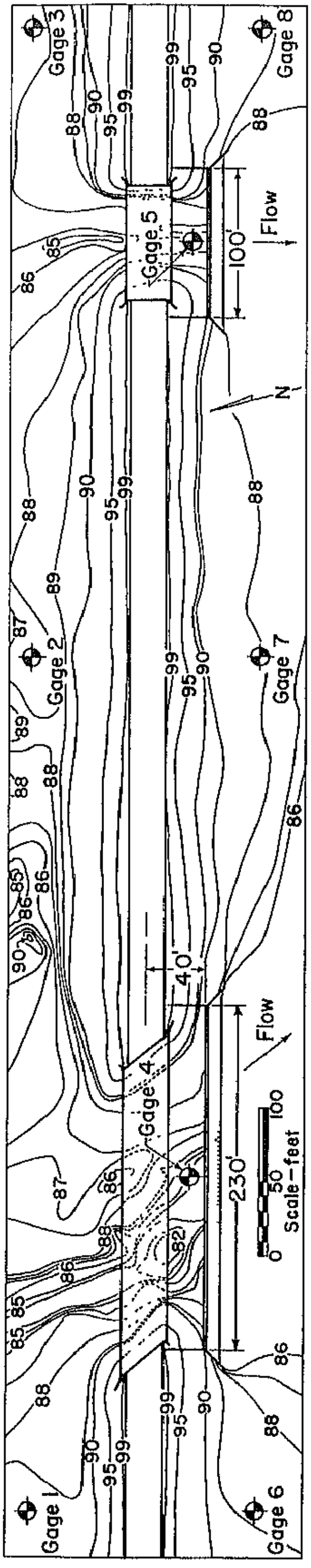


FIGURE 1.—Model topography and modifications at Gaging Station B, Little River Watershed near Tifton, Ga. (Elevations shown are referenced to benchmark on curb of west bridge, elevation 100.) Gage numbers refer to those used in the model.

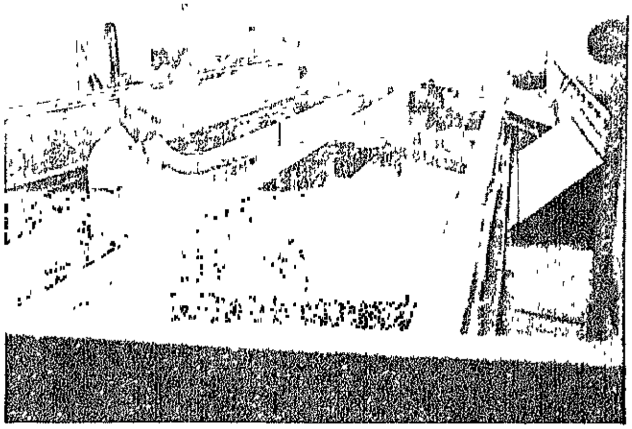


FIGURE 2.—Model construction of Gaging Station B, Little River Watershed (technician checking the level of the bridge).

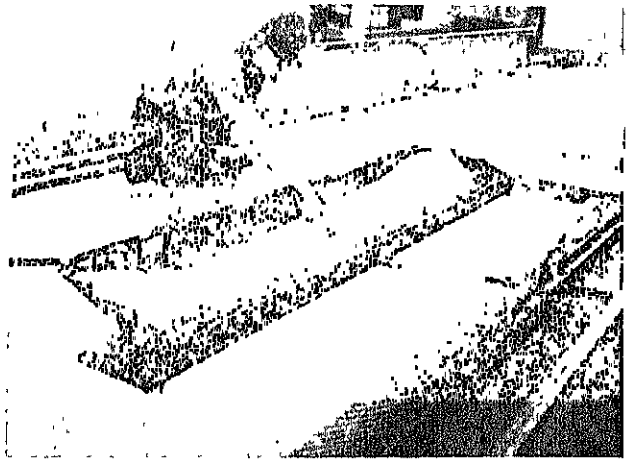


FIGURE 3.—Completed model (except for movable bed) of Gaging Station B, Little River Watershed (technician points out piezometer gage No. 4).

Brick-masonry sand was used for the movable bed. Scour of the movable bed material was used to evaluate the stilling-basin performance. This material was leveled under water to elevation 86 on the west bridge and elevation 88 on the east bridge prior to each erosion test. Grain diameters of 0.82, 0.47, and 0.27 mm were measured by sieve analysis for the 84.1 %, 50 %, and 15.9 % cumulative weights, respectively.

Support Equipment

The water supply for the model forebay was gravity fed through a 12-inch pipeline from Lake Carl Blackwell. Calibrated sharp-edged 0.875-, 1.5-, 2.5-, 4-, 6-, and 8-inch-diameter orifice plates inserted one at a time in the supply line were used to measure the flow rate, which was

controlled by an outlet valve. The discharge was determined from rating tables, using the differential head (measured to the nearest 0.1 inch of water) across the orifice. The water flowed from the forebay under pressure through a 1.5- by 2.0-ft box culvert with an adjustable orifice opening at the top along the wall of the basin. This orifice opening along the full length of the approach to the model was adjusted to produce a level water surface between gages 1, 2, and 3 (fig. 1) with a free design prototype discharge of 18,000 ft³/s (estimated 500-yr return period). A floating wave suppressor and baffles were used in the 2-ft approach between the orifice opening along the wall and the model.

The outflow from the model was through an adjustable opening along the basin wall into a 1.5- by 1.5-ft box culvert, which had a 12-inch valve mounted in the downstream end to control the tailwater level. The outlet opening was adjusted to maintain a level water surface between tailwater gages 6 and 8 (fig. 1) for the design flow of 18,000 ft³/s (prototype).

Water level recorders were installed in stilling wells in the forebay and in the tailwater area. These were used as visual indicators of steady flow in the model. The heads (nearest 0.001 ft) were measured in eight point gage wells mounted on the basin wall. These wells were connected with a 0.25-inch pipe to each of the gages shown in figure 1.

STILLING-BASIN TESTS

A major objective of this study was to determine the protective works needed to prevent damaging scour below the weir, scour that could also threaten the stability of the bridge. An apron with an end sill to create a shallow stilling basin and floor blocks to aid in energy dissipation on the apron were selected to provide the needed protection. Preliminary tests were run on floor-block and end-sill configurations to find a satisfactory arrangement. An arrangement that showed promise in the preliminary tests was subjected to more intensive testing to make certain that it would be satisfactory for the full range of flow conditions in the field. The final design which evolved from these tests is shown in figure 4. Only the tests on the final design are reported.

The protective works chosen consist of aprons extending 10 ft outward for the full length of the weirs. The elevations of the aprons are at the average natural ground elevations below the bridges, 88.0 and 86.0 ft for the east and west bridges, respectively. Floor blocks downstream of the V-notch portion of the weir are 9-inch cubes. Their arrangement is shown in figure 4. A 6-inch-high flip sill at the end of the apron extends for the full width of the apron except for the 3-ft openings at each end which provide for drainage of the basin.

The training walls for the prototype are to be constructed of MP-115 steel sheet piling. They will be placed perpendicular to the roadway up to the weir and flared at 45° to the end of the apron, as shown in figure 4. Each succeeding pile in the flared section is reduced 1 ft in height in the downstream direction (fig. 5). These details were carefully reproduced in the model, including the configuration of the sheet piling.

Test Procedure

Before each test, the movable bed was filled and leveled to the elevation of each apron. The tailwater regulating valve was closed and water was added slowly to both downstream and forebay sides of the model to completely submerge the weirs. The test discharge was set and the tailwater was lowered rapidly to a predetermined level. The bed was allowed to erode for approximately 20 min (110 min prototype). Water surface elevations, discharge measurements and photographs were obtained near the end of the test. At the end of each test the tailwater valve and the inflow valve were closed simultaneously. The water was then drained slowly from the forebay and tailwater areas of the model. Photographs were taken and contours of the erosion pattern below each weir were recorded after each test.

Test Results

The stilling basin was tested over the range of discharge rates and tailwater conditions that are expected to occur in the field. The criteria for judging the performance of the stilling basin are the location, depth, and extent of erosion in the downstream channel. The depth and extent of erosion in the model are indicators of the erosion to be expected in the prototype, but not necessarily of its actual depth. However, the model test results provide a means for comparing various basins for performance and also indicate the probable effectiveness of a basin. The erosion tests are summarized in table 2.

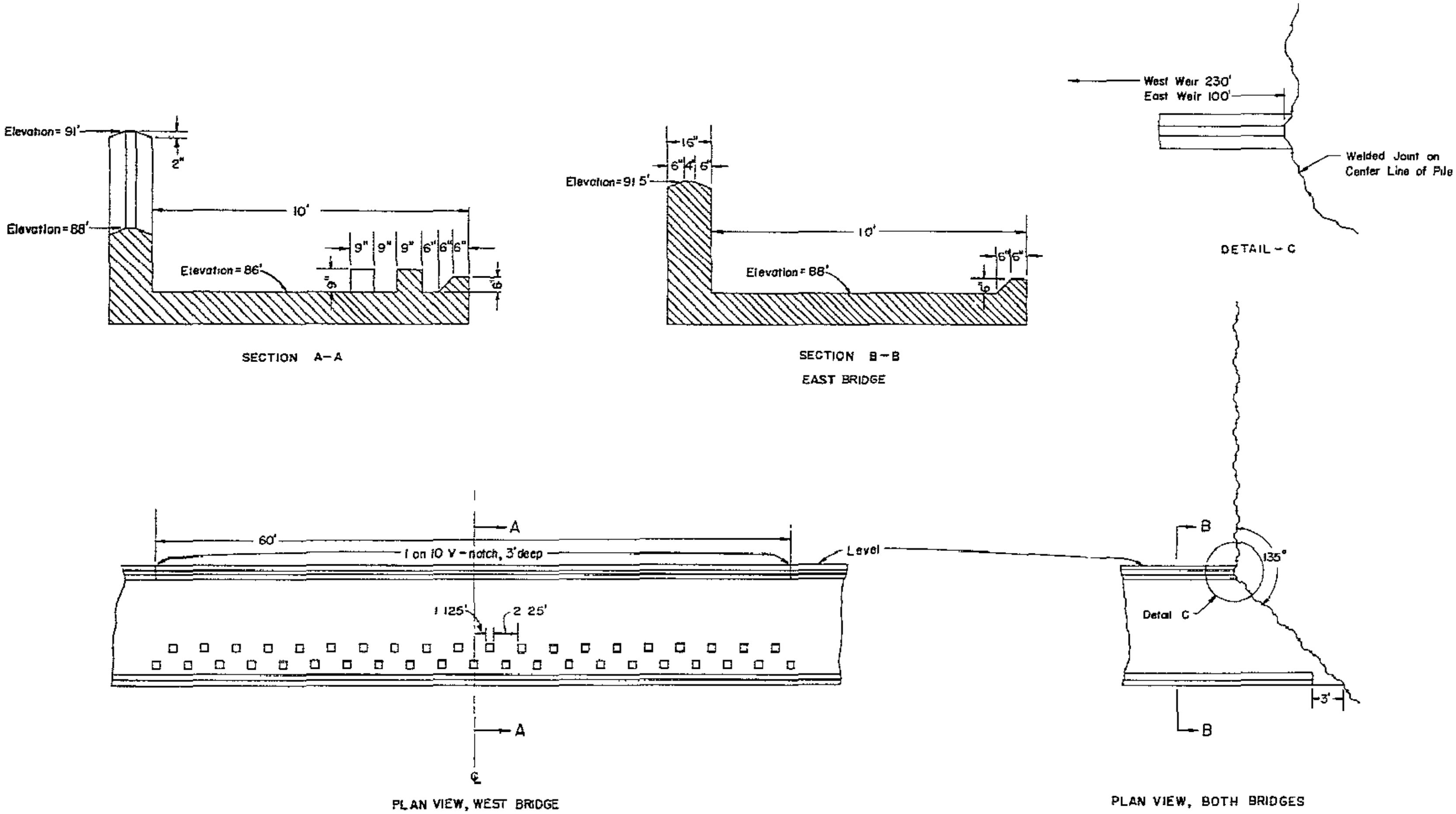


FIGURE 4.—Stilling basin for the flow-measuring weirs at Gaging Station B, Little River Watershed.

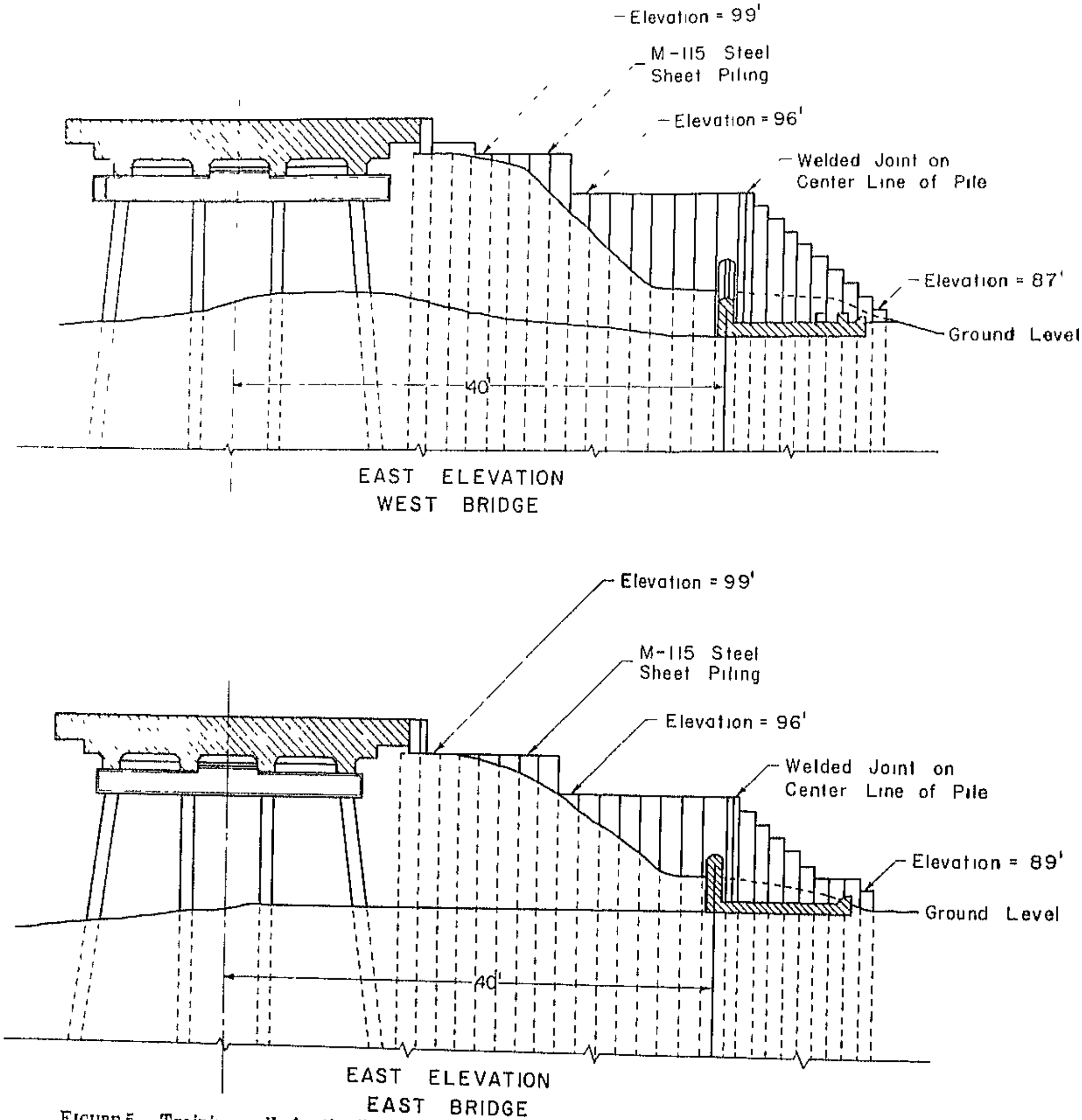


FIGURE 5.—Training walls for the flow-measuring weirs of Gaging Station B, Little River Watershed.

Various depths of water in the stilling basin were employed in the tests to determine the effect of this variable on the erosion. These depths are reported in the results as a submergence ratio T/H, where T is the average water surface elevation at downstream gages 6, 7, and 8, minus 88.0 (V-notch zero) and H is the average water surface elevation at upstream gages 1, 2, and 3, minus 88.0.

The scour pattern for a discharge of 152 ft^3/s is shown in figure 6. This test was run with a free outfall, the tailwater being established by the depth of flow over the sand bed. Maximum scour of 3.9 ft occurred 7 ft downstream of the end sill. The material removed formed an almost symmetrical delta downstream of the hole. The scour near the end sill was minor, with the maximum depth at 1.5 ft.

The flow was contained within the 1-in-10 V-notch weir for a discharge of 300 ft^3/s. The scour pattern for a submergence ratio of 12.3% (fig. 7) was narrow, with a maximum depth of 1.8 ft occurring 10 ft downstream of the end sill. The scour near the end sill was almost zero. Increasing the submergence ratio to 48.4% (fig. 8) reduced the area of scour. However, the maximum depth remained about the same or 1.7 ft.

A further increase in the submergence ratio

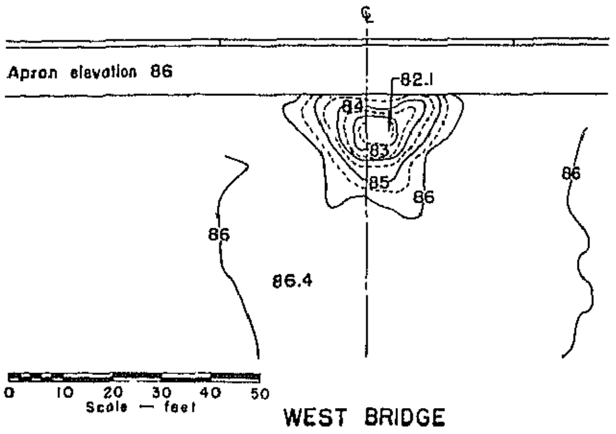
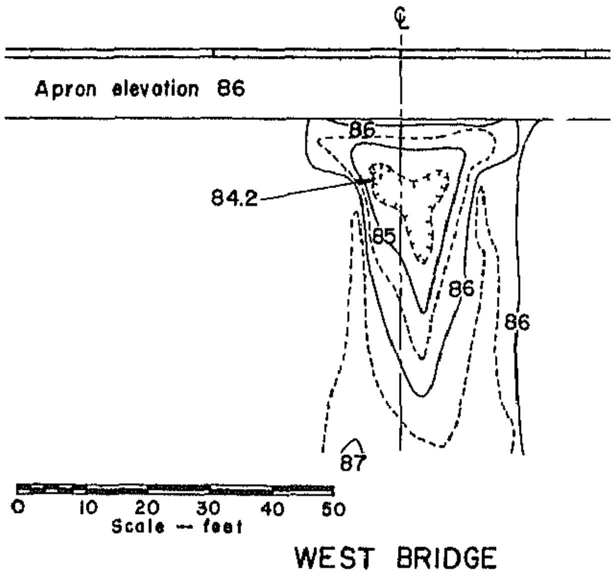


FIGURE 6.—Scour pattern after 126 min with a flow of 152 ft^3/s and submergence ratio less than zero.



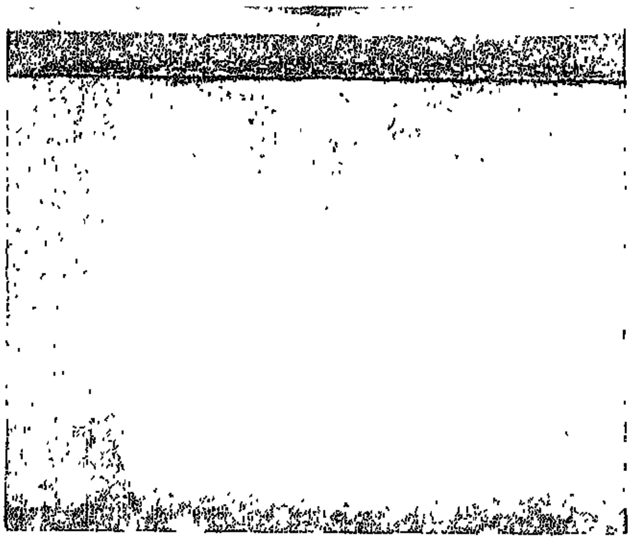


FIGURE 7.—Scour pattern after 110 min with a flow of 300 ft^3/s and submergence ratio of 12.3 %.

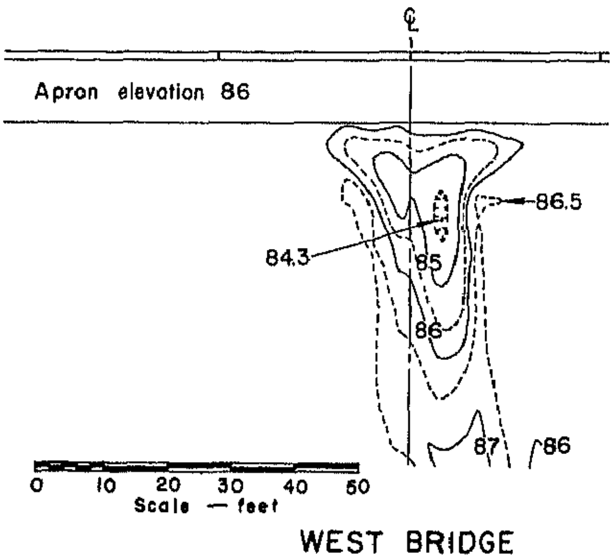


FIGURE 8.—Scour pattern after 110 min with a flow of 300 ft^3/s and a submergence ratio of 48.4 %.

TABLE 2.—*Summary of stilling-basin tests*

Discharge Q, ft³/s	Submergence ratio T/H, %	Maximum scour depth, ft	Scour time, min	Figure No.
152	...	3.9	126	6
300	12.3	1.8	110	7
300	48.4	1.7	110	8
300	69.6	0.95	110	9
1000	9.0	4.6	110	10
997	52.0	2.3	110	11
1005	69.6	2.6	115	12
4006	34.0	4.7	115	13, 14
4010	62.7	3.95	110	15, 16
4008	82.1	2.8	110	17, 18
8010	69.7	[1]8.27	110	19, 20
8015	78.6	4.0	120	21, 22
8010	84.4	5.2	115	23, 24

[1] Bottom limit of the basin.

to 69.6 resulted in a maximum scour depth of only 0.95 ft for a 110 min run (fig. 9).

The next tests were run at 1,000 ft³/s. At this rate all of the flow passed through the west bridge opening. Though some of this flow went over the horizontal crest weir, scour occurred only below the V-notch portion of the weir (fig. 10).

At a submergence ratio of 9.0% scour took place along the end sill at some places to a depth of 1.5 ft. The maximum depth of scour, 4.6 ft, occurred 20 ft downstream of the end sill. Increasing the submergence ratio to 52.0% produced a different pattern of scour (fig. 11). A single scour hole, which had a maximum depth of only 2.3 ft, developed. The sand bed near the end sill remained unchanged after the test. The maximum scour-hole depth for the 69.6% submergence ratio (fig. 12) was approximately the same as the scour hole for the 52% submergence ratio. However, the hole was closer to the end sill.

With total flow of 4,006 ft³/s, approximately 17% of the total flow was over the horizontal weir, downstream of the east bridge. The remaining 83% of the flow was over the V-notch and horizontal weirs downstream of the west bridge. For a submergence ratio of 34.0% (fig. 13) the maximum depth of scour for the west bridge was 4.7 ft and was located 21 ft downstream of the end sill. Aggradation occurred along the end sill of the west bridge weir except at the west end where 0.95 ft of degradation occurred. The maximum depth of scour at the east bridge weir was 4.6 ft and was located 8 ft downstream of the end sill (see fig. 14). Aggradation occurred along each end of the east bridge weir sill. Degradation just downstream of the end sill was less than 1 ft along the center portion of the sill.

Increasing the submergence ratio to 62.7% reduced the maximum scour depth from 4.7 ft to 3.95 ft downstream of the west bridge weir (fig. 15). The scour was almost eliminated downstream of the east bridge weir (fig. 16).

With a submergence ratio of 82.1% the maximum scour depth of 2.8 ft occurred near the west end of the bridge (fig. 17) and deposition occurred on the apron. Very little degradation occurred downstream of the east bridge (fig. 18).

With a total flow of 8,010 ft³/s, approximately 78% of the flow was over the weir of the west

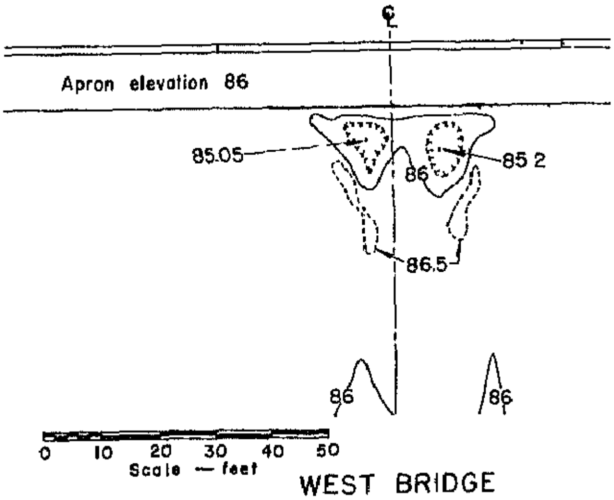
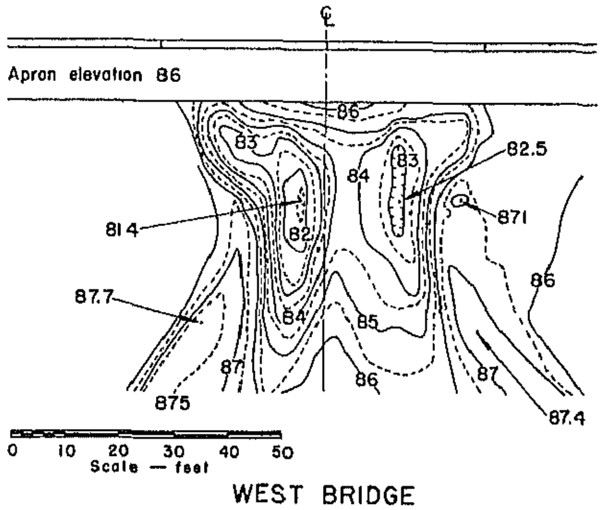


FIGURE 9.—Scour pattern after 110 min with a flow of 300 ft³/s and submergence ratio of 69.6%.



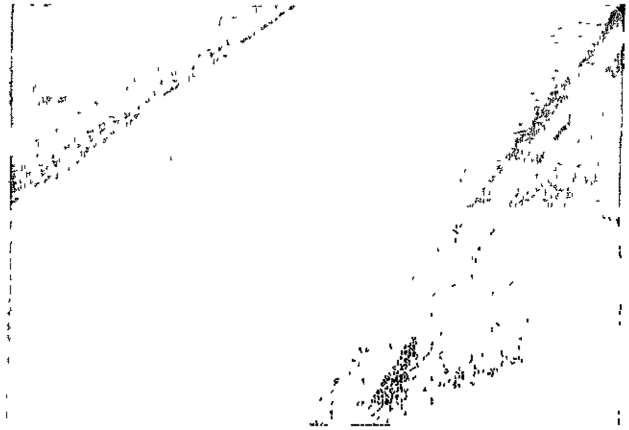
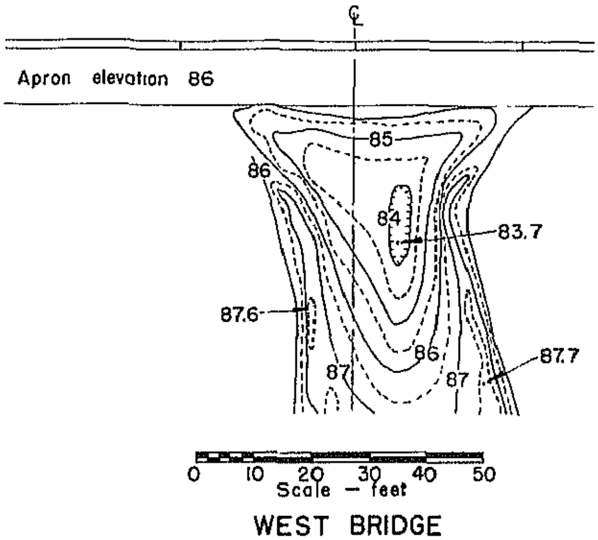


FIGURE 10.—Scour pattern after 110 min with a discharge rate of 1,000 ft³/s and a submergence ratio of 9.0%.



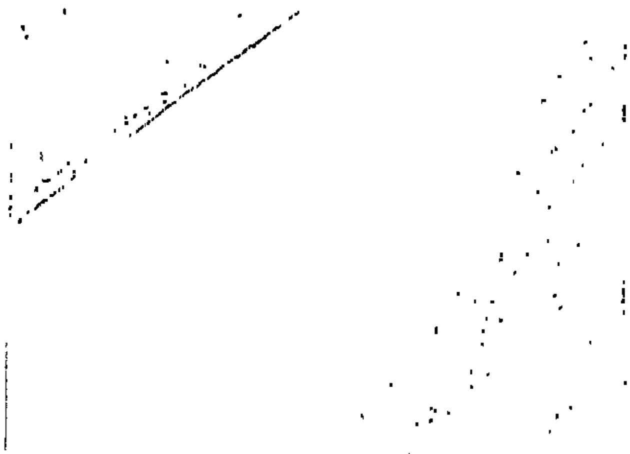
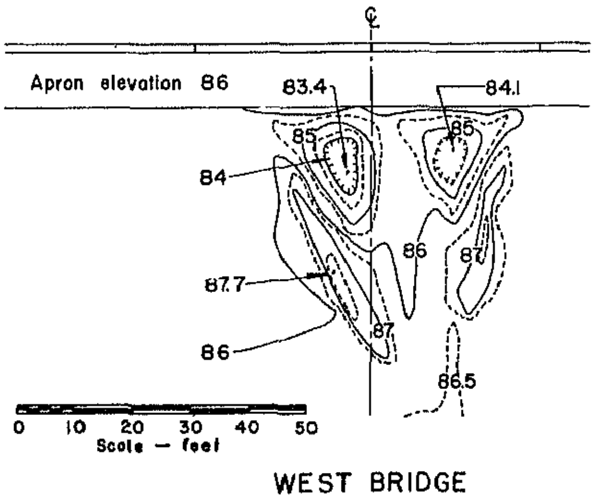


FIGURE 11.—Scour pattern after 110 min with a discharge rate of 997 ft³/s and a submergence ratio of 52%.



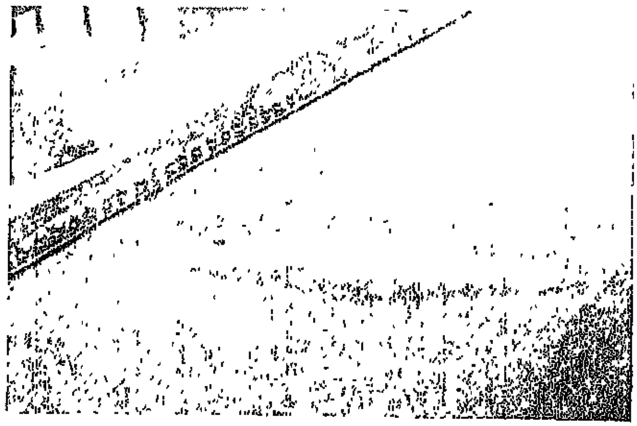


FIGURE 12.—Scour pattern after 115 min with a discharge rate of 1,005 ft³/s and a submergence ratio of 69.6%.

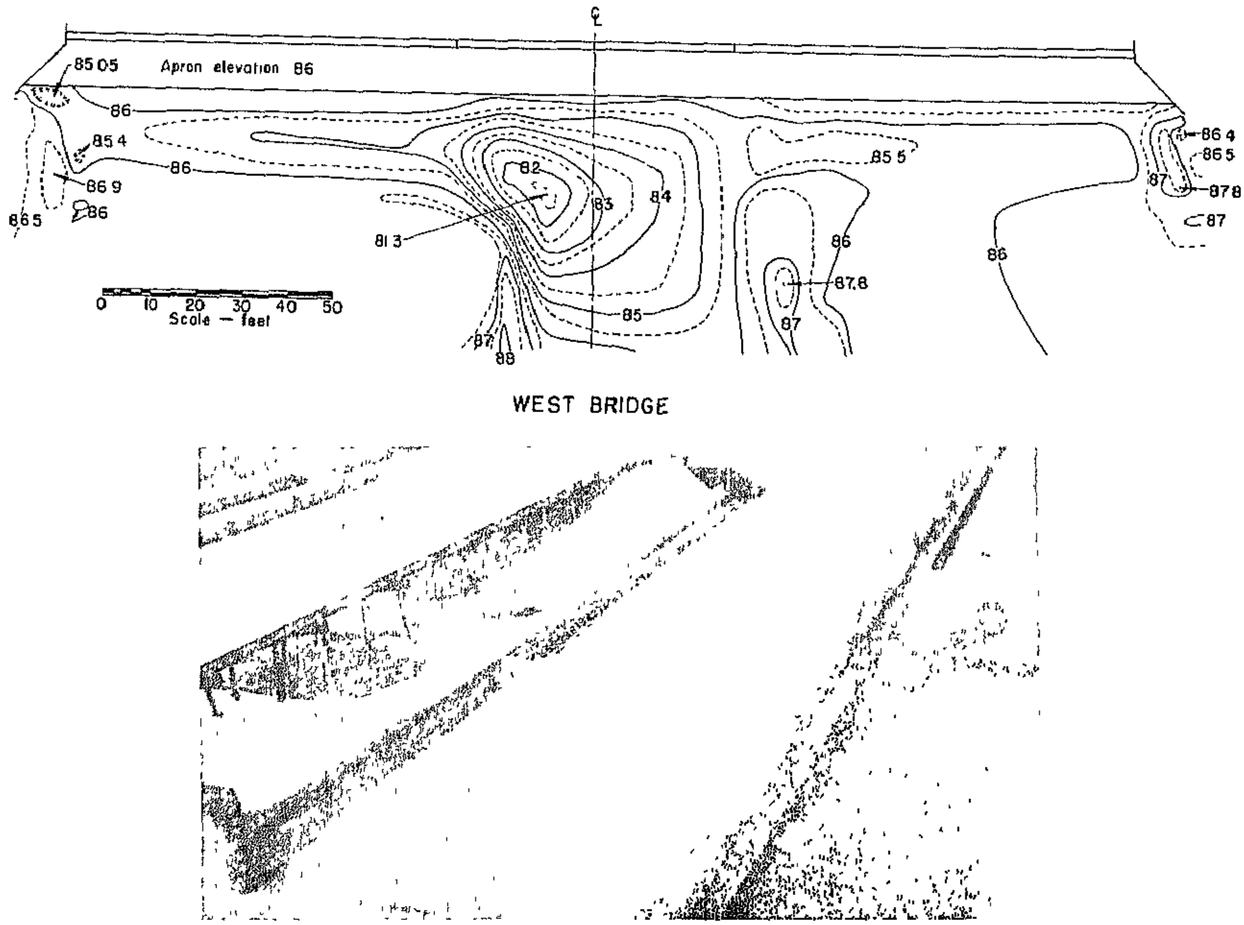
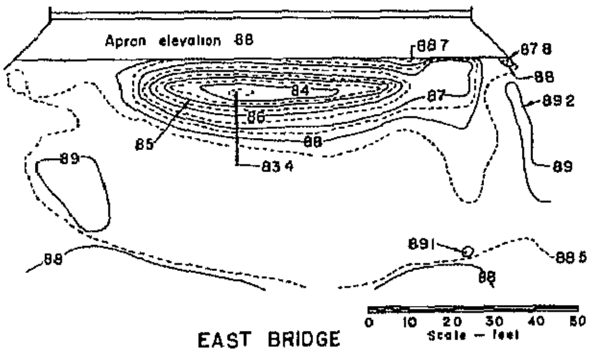


FIGURE 13.—Scour pattern for the west bridge after 115 min with a total discharge rate of 4,006 ft³/s and a submergence ratio of 34.0%.



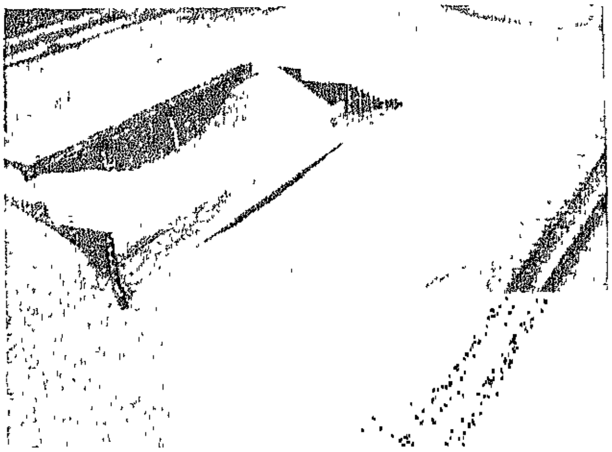


FIGURE 14.—Scour pattern for the east bridge after 115 min with a total discharge rate of 4,006 ft³/s and a submergence ratio of 34.0%.

bridge and the remainder over the east bridge weir. For a submergence ratio of 69.7% the flow beneath the west bridge scoured the erodible bed to the bottom limit of the basin, as shown by cross-hatching in figure 19. The maximum scour near the end sill was 5.5 ft. Degradation downstream of the east bridge was minor except for the east end, where a 3.8-ft depth occurred (fig. 20).

Increasing the submergence to 78.6% reduced the degradation downstream of the west bridge (fig. 21) to a maximum depth of 4.0 ft near the

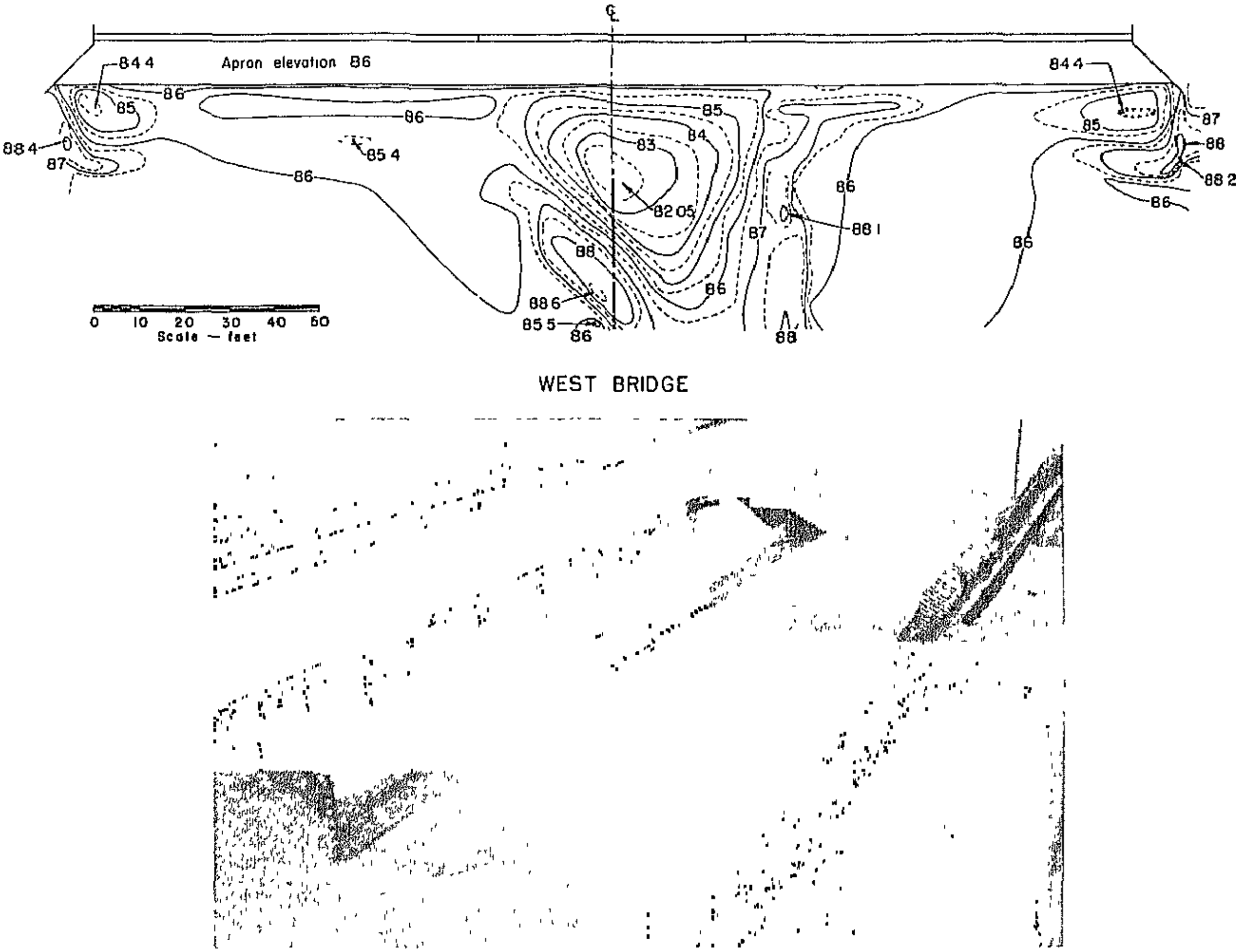
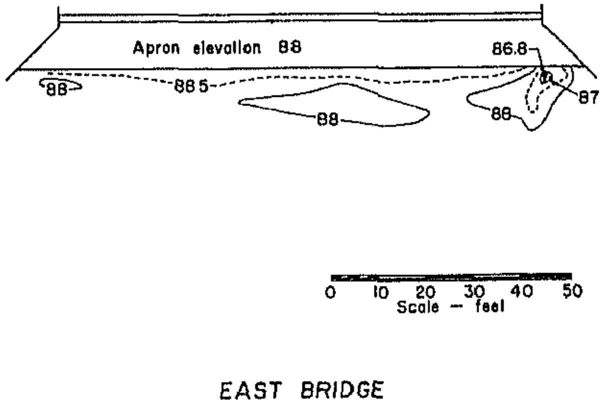


FIGURE 15.—Scour pattern for the west bridge after 110 min with a total discharge of 4,010 ft³/s and a submergence ratio of 62.7%.



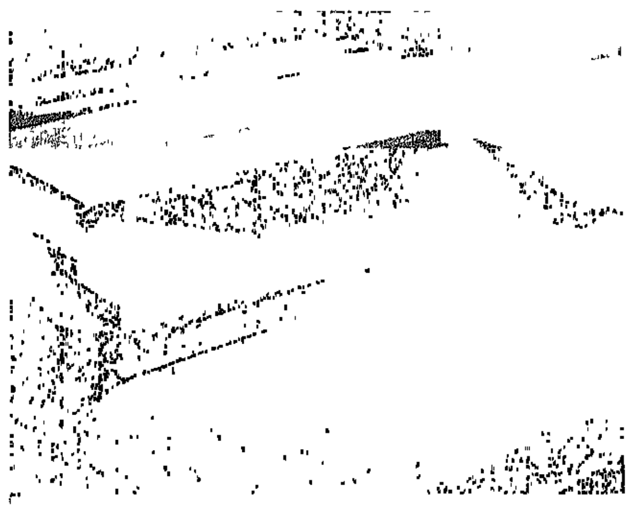


FIGURE 16.—Scour pattern for the east bridge after 110 min with a total discharge of 4,010 ft³/s and a submergence ratio of 62.7%.

west end of the weir. Material was deposited on the apron of the west bridge weir by return-flow circulation under the nappe. Degradation downstream of the east bridge was minor with a maximum depth of 3.1 ft occurring near the east end (fig. 22).

Increasing the submergence ratio to 84.4% reduced the scour downstream of the center portion of the west bridge (fig. 23). However, the scour was increased near the west end of the weir. Here, as at the east end, the nappe was plunging into the basin without an undular jump

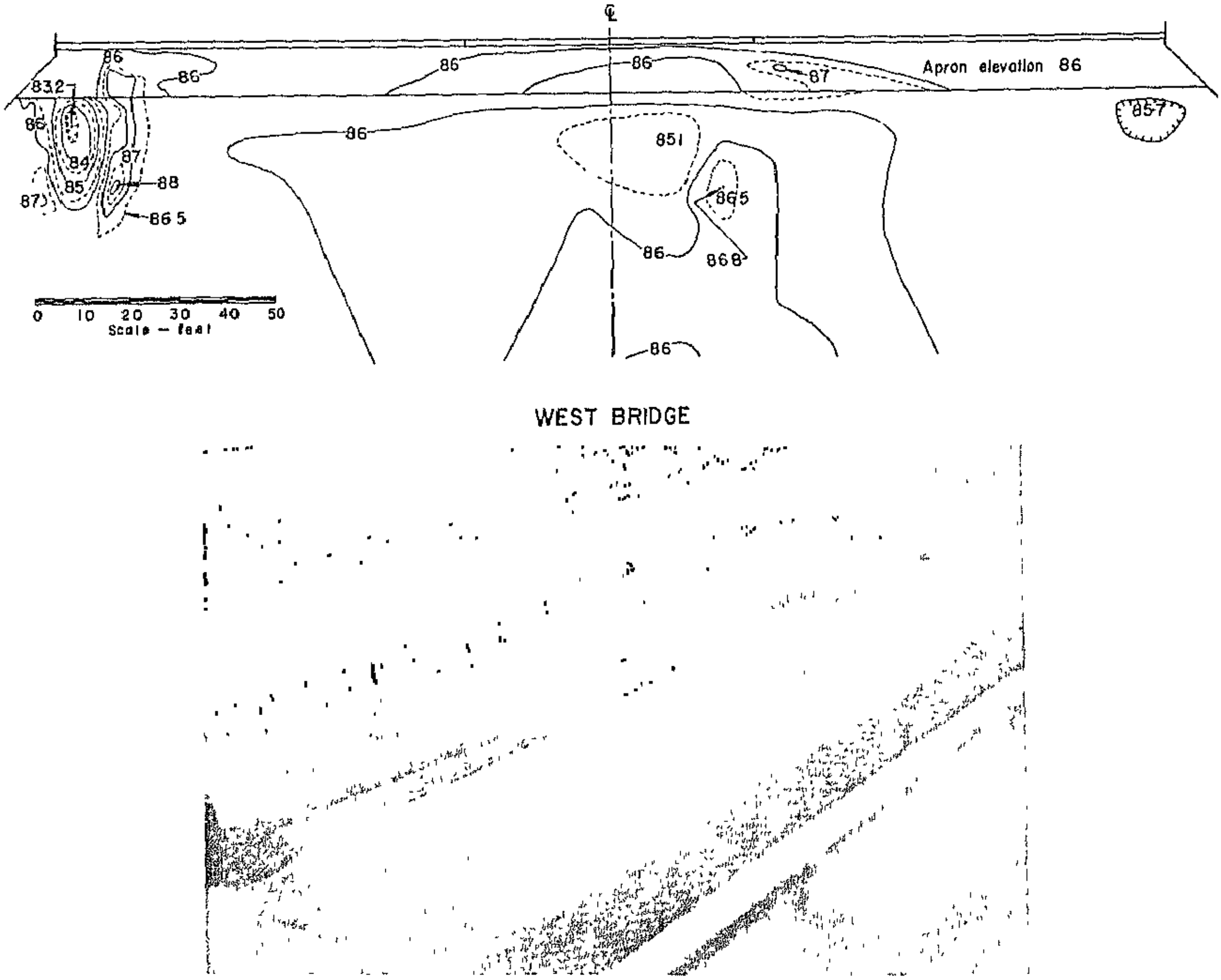


FIGURE 17.—Scour pattern for west bridge after 110 min with a total discharge of 4,008 ft³/s and a submergence ratio of 82.1%.

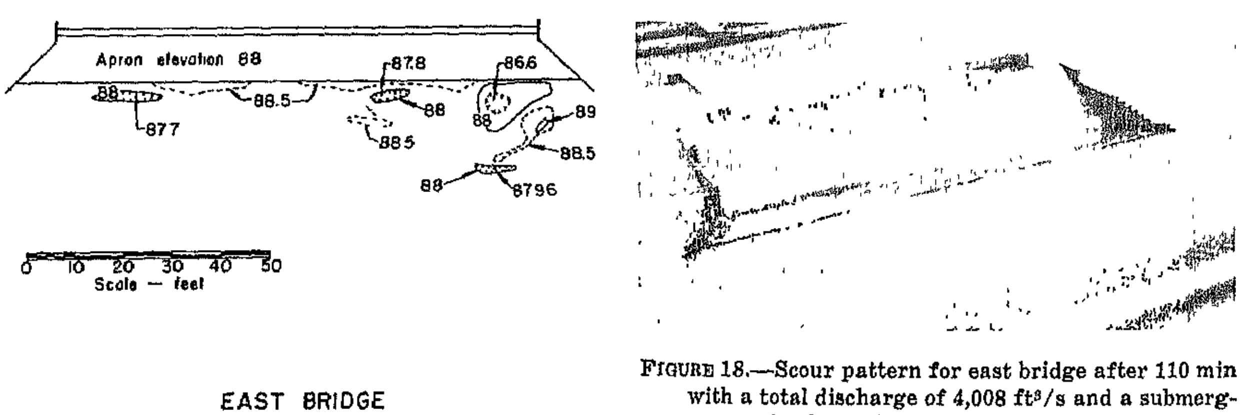


FIGURE 18.—Scour pattern for east bridge after 110 min with a total discharge of 4,008 ft³/s and a submergence ratio of 82.1%.

forming (see photograph in fig. 23). This same plunging nappe occurred in the stilling basin of the east bridge and produced the maximum scour depths at the ends (fig. 24).

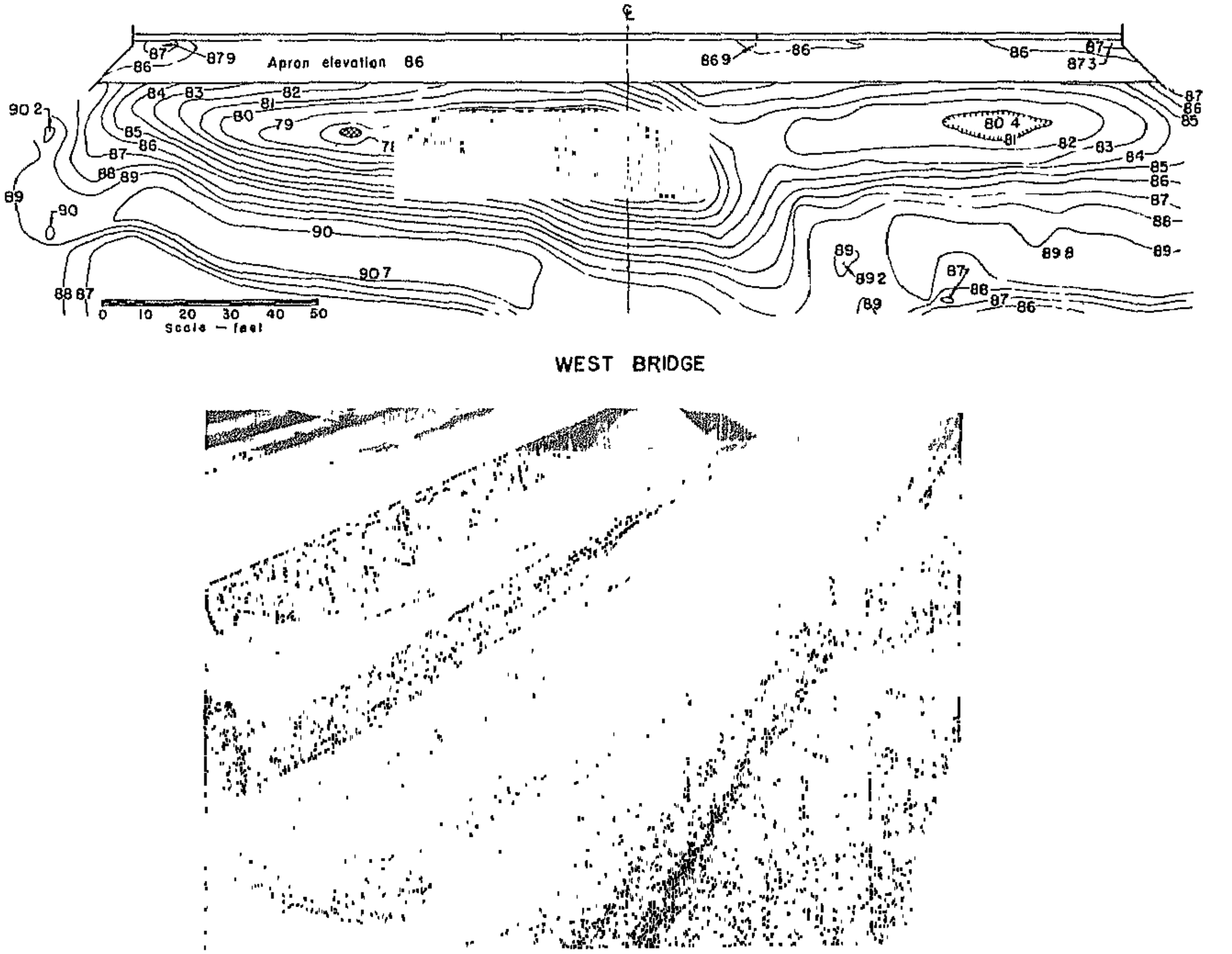


FIGURE 19.—Scour pattern for west bridge after 110 min with a total discharge of 8,010 ft³/s and a submergence ratio of 69.7%.

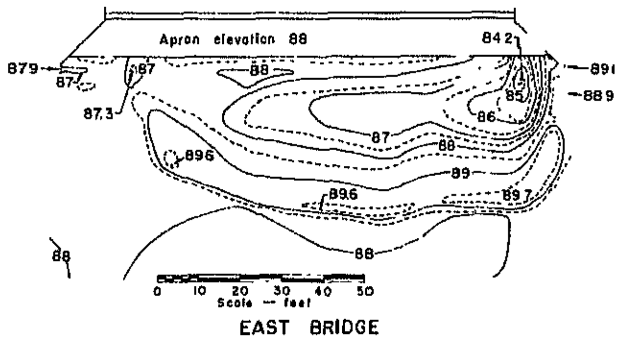
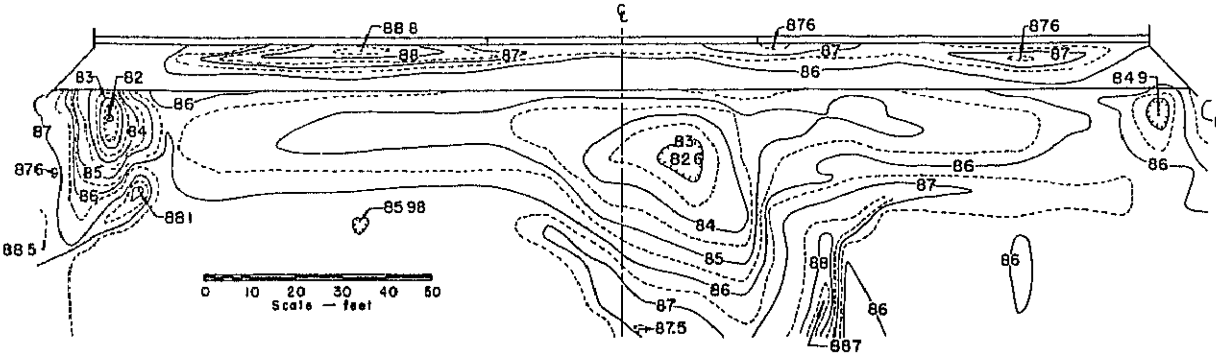


FIGURE 20.—Scour pattern for east bridge after 110 min with a total flow of 8,010 ft³/s and a submergence ratio of 69.7%.



WEST BRIDGE

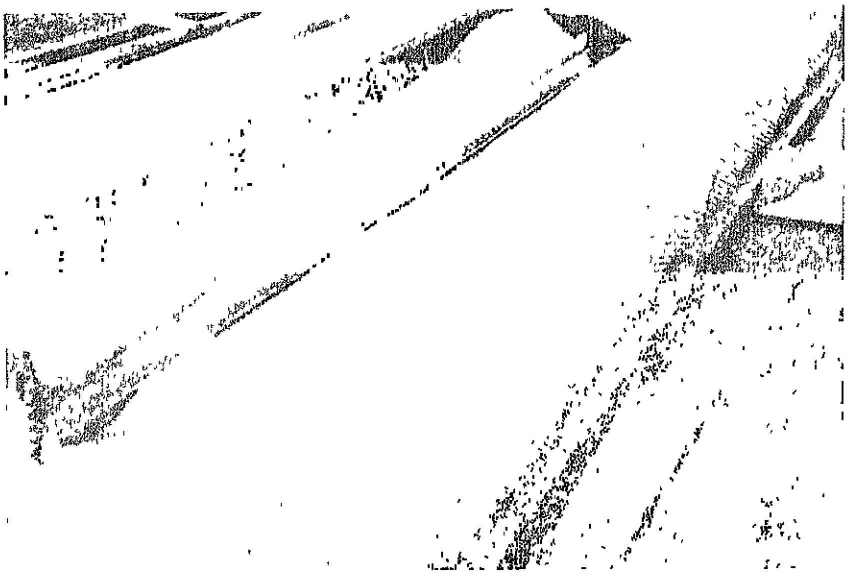
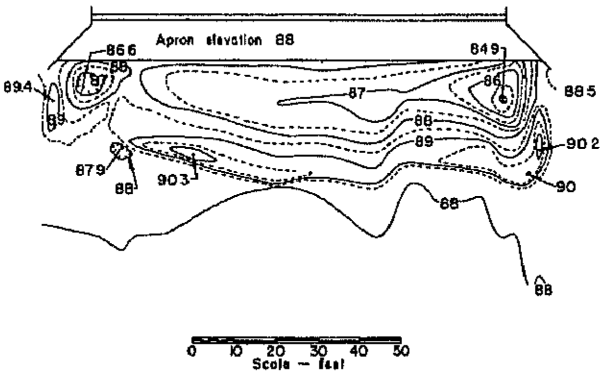


FIGURE 21.—Scour pattern for west bridge after 120 min with a total discharge of 8,015 ft³/s and submergence ratio of 78.6%.



EAST BRIDGE

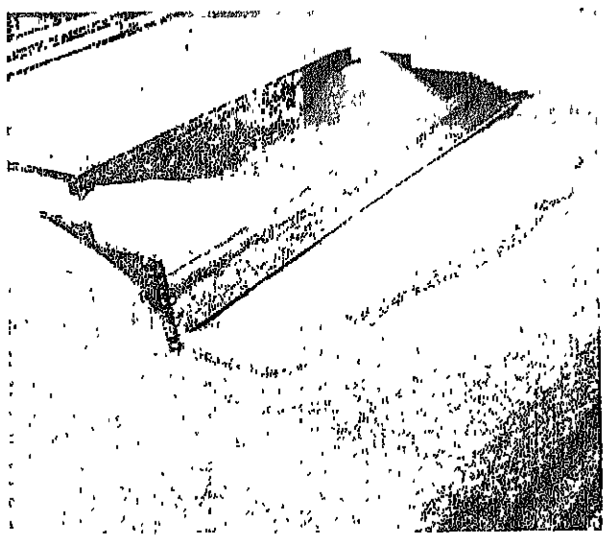
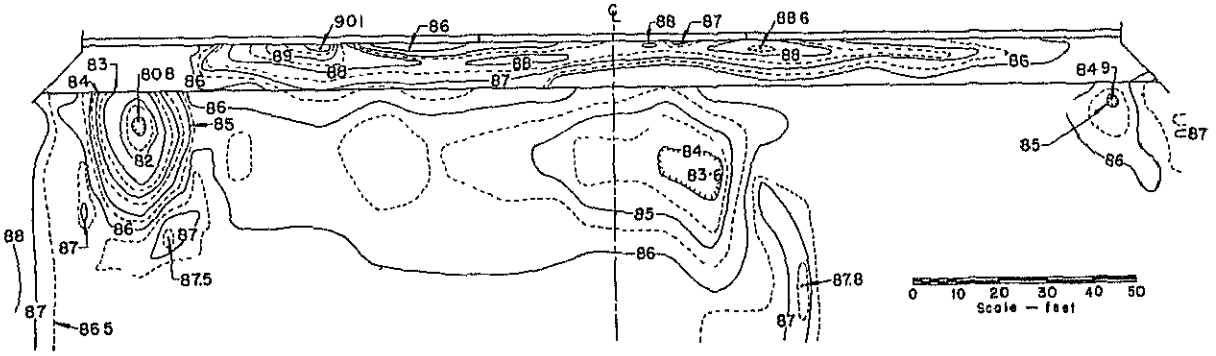
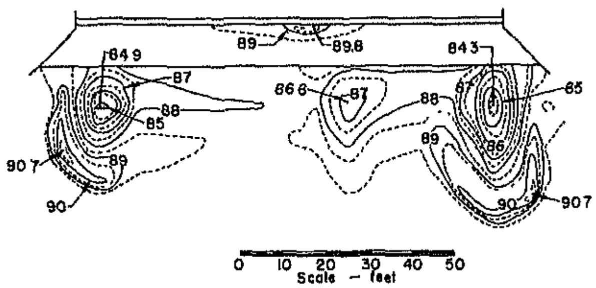


FIGURE 22.—Scour pattern for east bridge after 120 min with a total discharge of 8,015 ft³/s and submergence ratio of 78.6%.



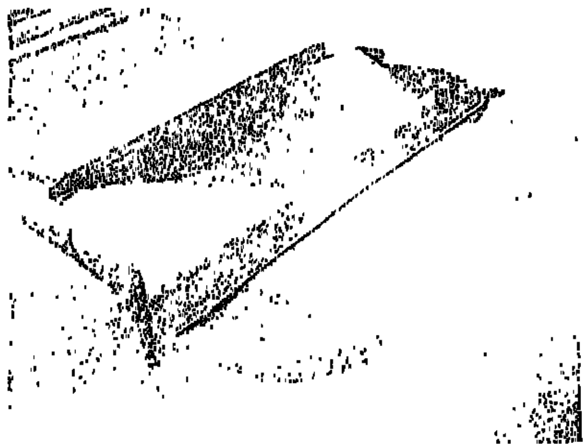
WEST BRIDGE

FIGURE 23.—Scour pattern and flow conditions for west bridge after 115 min with a total discharge of 8,010 ft^3/s and a submergence ratio of 84.4%.



EAST BRIDGE

FIGURE 24.—Scour pattern for east bridge after 115 min with a total discharge of 8,010 ft^3/s and a submergence ratio of 84.4%.



Minimum observed scour hole elevations in feet for the west bridge are plotted for all tests in figure 25. Contours of equal scour hole depths are also drawn in this figure. The figure shows that scour depth increased with increasing discharge and decreased with increasing submergence. Except for the test at 8,010 ft^3/s and a submergence of 69.7%, which caused a scour depth of over 8 ft, all other flows caused depths of scour less than 5 ft. However, because of existing downstream vegetative conditions, the expected submergence will be greater than 85% for a discharge rate of 8,010 ft^3/s, so the lack of sufficient sand depth in the model at a submergence of only 69.7% for 8,010 ft^3/s is not a matter for concern. This combination of discharge and submergence lies outside the probable range in the field.

HEAD-DISCHARGE RATING

Determination of the head-discharge rating was the second objective of the model study. It was already known from the tailwater studies and the stilling-basin tests that the weirs would operate as submerged weirs for many of the flows and the rating would have to take this into account. Therefore, the general approach was to rate the weirs for free discharge and for various degrees of submergence. The free-flow ratings will be described first.

Free-Flow Rating

The usual head measuring location for a broad-crested weir is 10 ft upstream of the weir. This is the location of gages 4 and 5 (fig. 1). However, since the bridges could exercise control at the higher flows, gages 1 and 2 were selected for the headwater measurement. As shown in figure 1, these gages are located on each side of the approach to the west bridge. Headwater measurements were also made at gages 4 and 5 for comparison with gages 1 and 2 and for analysis of head-discharge relationships for the weir alone.

Surface-tension effects are not evaluated with the Froude model law. Therefore, low flows that are affected by surface tension in the 1:30 model cannot be used for a rating. Full-scale sections of the weir were used to evaluate discharge coefficients for these flow depths. The results of the tests on the full-scale sections are found in Appendixes A and B.

For the free-flow rating the tailwater gate was left open and the flow was controlled by the weirs and bridges. The following equation was used in the analysis of the data:

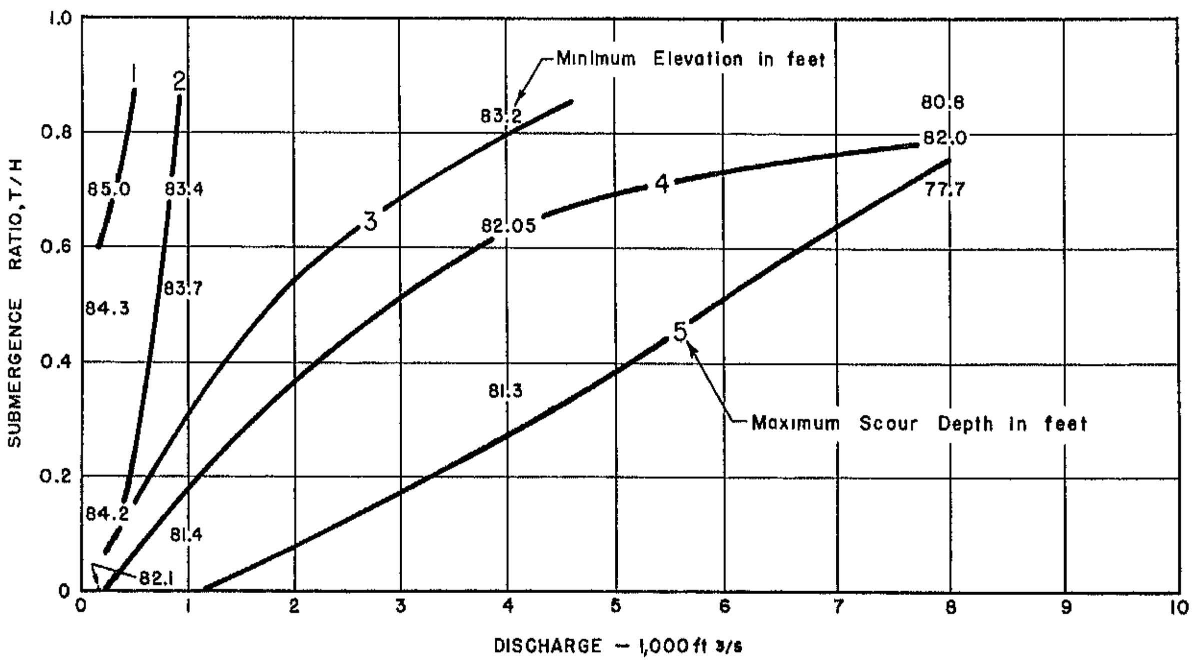


FIGURE 25.—Maximum scour depths in feet (after approximately 110 min of flow) occurring downstream of the west bridge of site B model.

$$Q = C[\Sigma L_i H_i^{1.5} + (\tan \Theta/2) H_v^{2.5}], \qquad (1)$$

where Q = total discharge (cubic feet per second),
C = discharge coefficient (feet $^{0.5}$ per second),
L_i = individual lengths of horizontal crest (feet),
H_i = individual head above horizontal crest (feet),
Θ = total angle (degrees) of the V-notch crest,
and H_v = head (ft) above the V-notch, measured by an average of gages 1 and 2 above the intercept of the two crests.

The term $(\tan \Theta/2) H_v^{2.5}$ in equation 1 is the general expression for free surface flow through a V-shaped opening, apex down. Its derivation is based on the area of the triangle which is a function of the head and the slopes of the sides. When the head exceeds the depth of the V-notch the outer ends of the calculated triangle extend over the adjoining horizontal crests. To include the full length of the horizontal crests in the first term within the brackets of equation 1 would include the overlapped portion of the horizontal crests twice in the equation. The net effect of doing this would be the introduction of more variation in the coefficient C. Therefore, in an effort to keep C more nearly constant the length of the horizontal crest on each side of the V-notch weir of the west bridge was reduced by $(\tan \Theta/2)$ $H_i/2$. This correction factor is equal to one-half the length of the part of the horizontal crest overlapped by the calculated triangle. The correction factor agrees reasonably well with an analytically derived expression and is much easier to use.

The results of the test are presented in figure 26. The H_v values were obtained from an average of the water stage elevation at gages 1 and 2. There is some scatter in the model data points with H_v values less than 5 ft. Some of this is due to surface-tension effects present for the low flows $(3.0 < H_v < 4.2)$ over the long horizontal weirs. Water surface elevations for gages 4 and 5 of the model were used along with the full-scale coefficients (see Appendixes A and B) to compute the discharge in this range. The resulting computed discharge coefficients for H_v measured at gages 1 and 2 are shown as solid points and dashed lines in figure 26.

Submergence Rating

The data for the submergence rating were obtained at the same time the stilling basin tests were run. Additional data were necessary to complete the full range of submergence for each discharge rate.

An average of the water surface elevations for gages 1 and 2 (fig. 1) above the V-notch zero (elevation 88.0) was used for the head H_v. The tailwater elevation above the V-notch zero T was obtained from an average of gages 6, 7, and 8 (see fig. 1). Thus, the head loss for the bridges and weirs combined is measured by the difference in the average water level of gages 1 and 2 and the average water level of gages 6, 7, and 8. The test procedure was to set a constant flow and starting with complete submergence or a high tailwater, to lower the tailwater in steps. After each decrease the H_v and T elevations were allowed to become steady before readings were recorded. The method employed to analyze the submergence data for the full size horizontal crest weir (Appendix B) was used to analyze these data. The results are presented in figure 27.

Two relationships are evident in this plot. The lower line represents the flow contained wholly within the V-notch, the 150 ft^3/s and the 300 ft^3/s flows in these tests. The upper line represents flows which were over both the horizontal and the V-notch weirs. The two empirical curves shown in figure 27 were obtained using the method of least squares, in which $Y = \log_{10}(1 - Q/Q_F)$ and $X = \log^{10}(e^{T/H})$.

The following equation was obtained for flows within the V-notch portion of the weir:

$$Q/Q_F = 1 - 7.521 \times 10^{-3} [e^{T/H_v}]^{1.498}, \qquad (2)$$

where Q = actual discharge (cubic feet per second) for submerged flow,
Q_F = free discharge (cubic feet per second) for equivalent head,
e = 2.7183 . . . ,
T = tailwater elevation above V-notch zero (elevation 88.0),
and H_v = head (equal to or less than 3.0 feet) at gages 1 and 2.

The following equation was obtained for flows through the V-notch and over the horizontal crest portions of the weir:

$$Q/Q_F = 1 - 6.326 \times 10^{-1} [e^{T/H_v}]^{0.395}, \qquad (3)$$

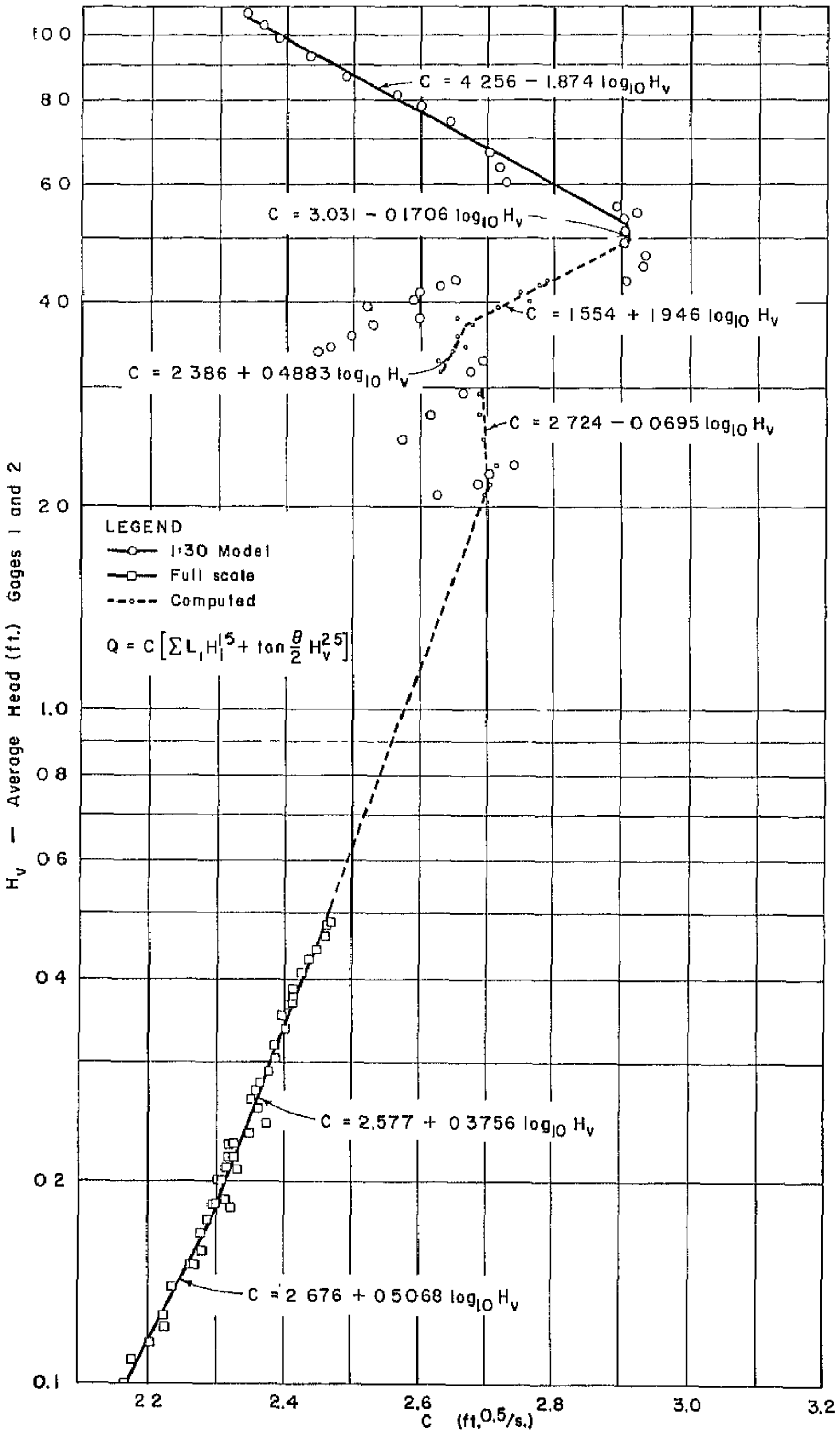


Figure 26.—Discharge coefficients for free-flow rating of Little River Gaging Station B.

where H_r=head (greater than 3.0 ft) at gages 1 and 2 above V-notch zero (elevation 88.0).

RESULTS

The stilling basin developed in the model studies for the weirs is a level apron extending 10 ft out from the weir for its full length (fig. 4). A 6-inch-high end sill is at the downstream edge of the apron. Cubical blocks are placed on the apron below the V-notch portion of the weir at the main opening. The deepest scour observed in the channel downstream from the V-notch weir was generally less than 5 ft for the range of discharges and related submergences expected in the field.

The low-flow control at the west bridge opening across the main stream is a 1-on-10 V-notch, broad-crested weir 3 ft deep, flanked by horizontal weirs each 85 ft long. The control across the overflow opening at the east bridge is a horizontal weir set 0.5 ft higher than the horizontal weirs at the main stream. Seventy percent of the average annual flow volume will be contained wholly within the V-notch. Because of the low stream gradient and the impracticability of raising the weirs, high tailwater will affect weir performance at the larger flows. Therefore, submergence ratings were developed.

The lengths of the approach and exit sections of the model were limited and there was the pos-

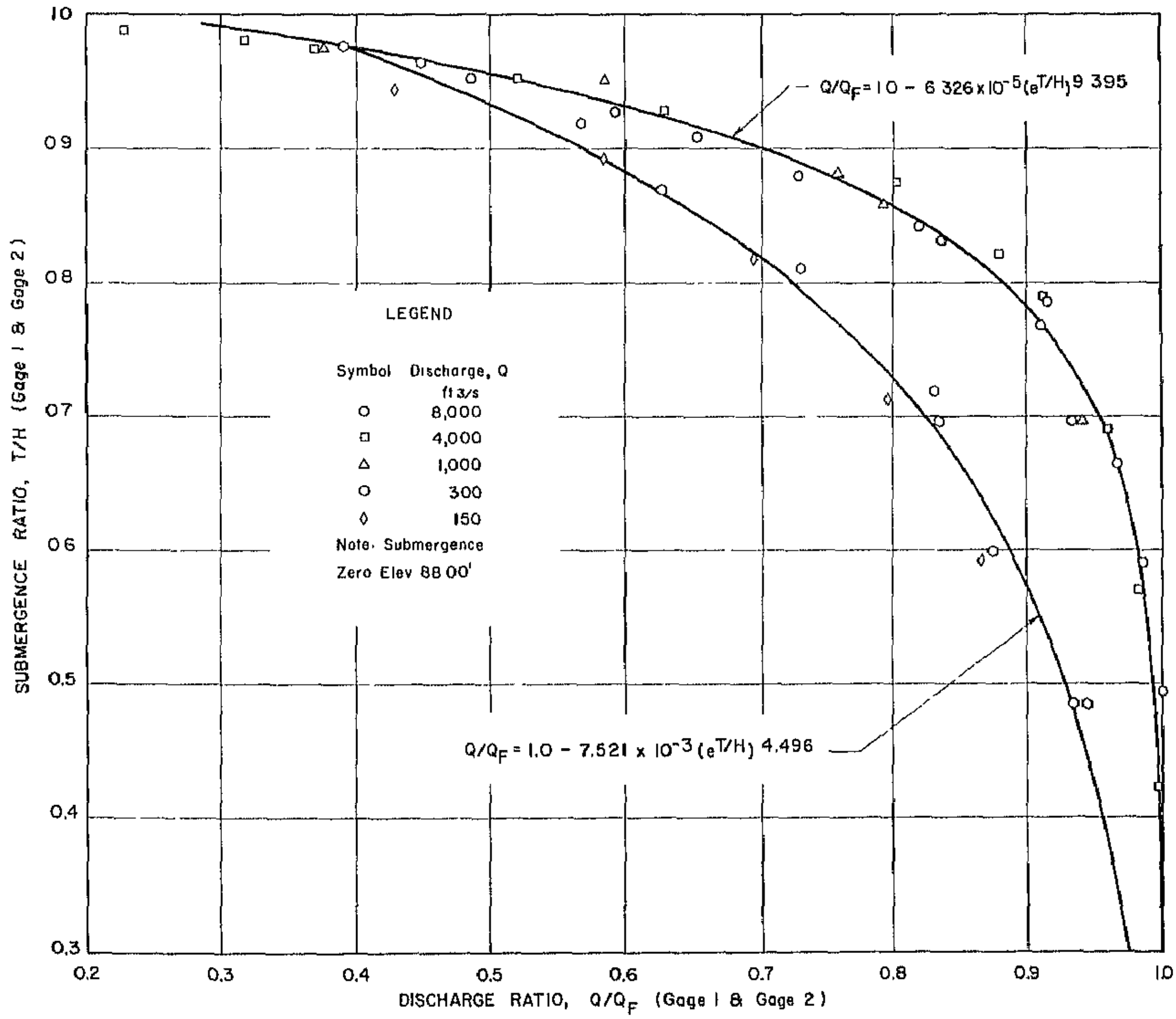


FIGURE 27.—Discharge-submergence relationship for Little River Gaging Station B.

sibility that the water surface elevations in the gage wells would not reproduce exactly the field values at these same locations. Therefore, averages of water surface elevations measurements at two points upstream of the bridges and three points downstream of the bridges were used to determine the headwater and tailwater levels respectively in the model. In the field, if the water surface elevations in the gage locations is level across a section transverse to the flow and is not unduly disturbed, fewer points of measurement can be used. However, a minimum of two gage locations will be necessary during submerged conditions, one upstream of the bridges and one downstream of the weir, preferably at points of approximately zero velocity. The submergence ratings developed in this report should be considered preliminary and approximate. Discharge ratings should be obtained in the field as opportunities occur to develop a reliable submergence rating.

APPENDIX A

Full-Scale Rating for the 16-Inch 1-on-10 V-Notch Weir

A full-scale, concrete, broad-crested weir was constructed in the laboratory basin. (Its design is shown in figure 28.) This basin is the same one used for the model tests and has a capacity of 4 ft^3/s. Therefore, the head on the V-notch was limited to 0.5 ft. A crib baffle around the 12-inch water supply line and a float were used to produce a symmetrical and undisturbed flow.

The following formula was used to analyze the weir tests:

$$Q = C_v (\tan \theta/2) H_v^{2.5}, \qquad (4)$$

where Q = discharge (cubic feet per second),
C_v = discharge coefficient (feet$^{0.5}$/second),
θ = total angle (degrees) of the V-notch crest,
and H_v = head (ft) above the V-notch, measured from the intercept of the two crests.

The results of the tests for free outflow are presented in figure 29 and table 3. The equations for C_v were obtained using the method of least squares and the log $_{10}$ transform of H_v.

APPENDIX B

Full-Scale Rating for the 16-Inch Horizontal Broad-Crested Weir

The same crests used in the full-scale V-notch test were reset in the basin to form a horizontal roroad-crested weir 8 ft long as shown in figure). A 1-ft offset with a parabolic approach form as used at each end of the weir to eliminate end ontraction effects. Another weir length, 4 ft, as also tested with a parabolic approach to determine if end contraction effects were actually eliminated. If so, the data would represent a unit length of weir and could be used to evaluate the discharge for low flows over the long, horizontal weirs of Gaging Station B.

These data were analyzed using the following weir formula:

$$Q = C_F L H^{1.5}, \qquad (5)$$

where Q = discharge (cubic feet per second),
C_F = discharge coefficient for horizontal weir (feet$^{0.5}$/second),
L = weir length (feet),
and H = head above crest (feet).

The coefficients are plotted versus head in figure 31 for free outflow. The coefficient values for a given head are approximately the same for either the 8.005-ft- or the 3.950-ft-long weir. Therefore, end contraction effects had been eliminated. The equations in figure 31 were obtained by the method of least squares. The data show a definite change in the equation at heads near 0.2 ft and 0.03 ft. These same indications of changes may be seen in figure 29 for the V-notch tests.

Submergence of Horizontal Crest

Submergence tests were made by setting a constant flow, increasing the tailwater level and measuring the tailwater T and the head H above the crest. The locations of the gages are shown in figure 30. After each increase in tailwater level the flow was allowed to become steady before readings were recorded.

The ratio of tailwater head T to headwater head H was calculated for each submergence test. Then the ratio of the actual discharge during the

submergence test to the free outfall discharge for the same head H was calculated. The relationship between corresponding values of the two ratios, T/H and Q/Q_F, is shown in figure 32. For ratios of T/H of less than 0.6 the ratio Q/Q_F was unity, i.e., the upstream head H was unaffected by tailwater when T was less than $0.6H$. Plotting indicated two relationships, one for heads less than 0.29 ft and another for heads equal to or greater than 0.29 ft.

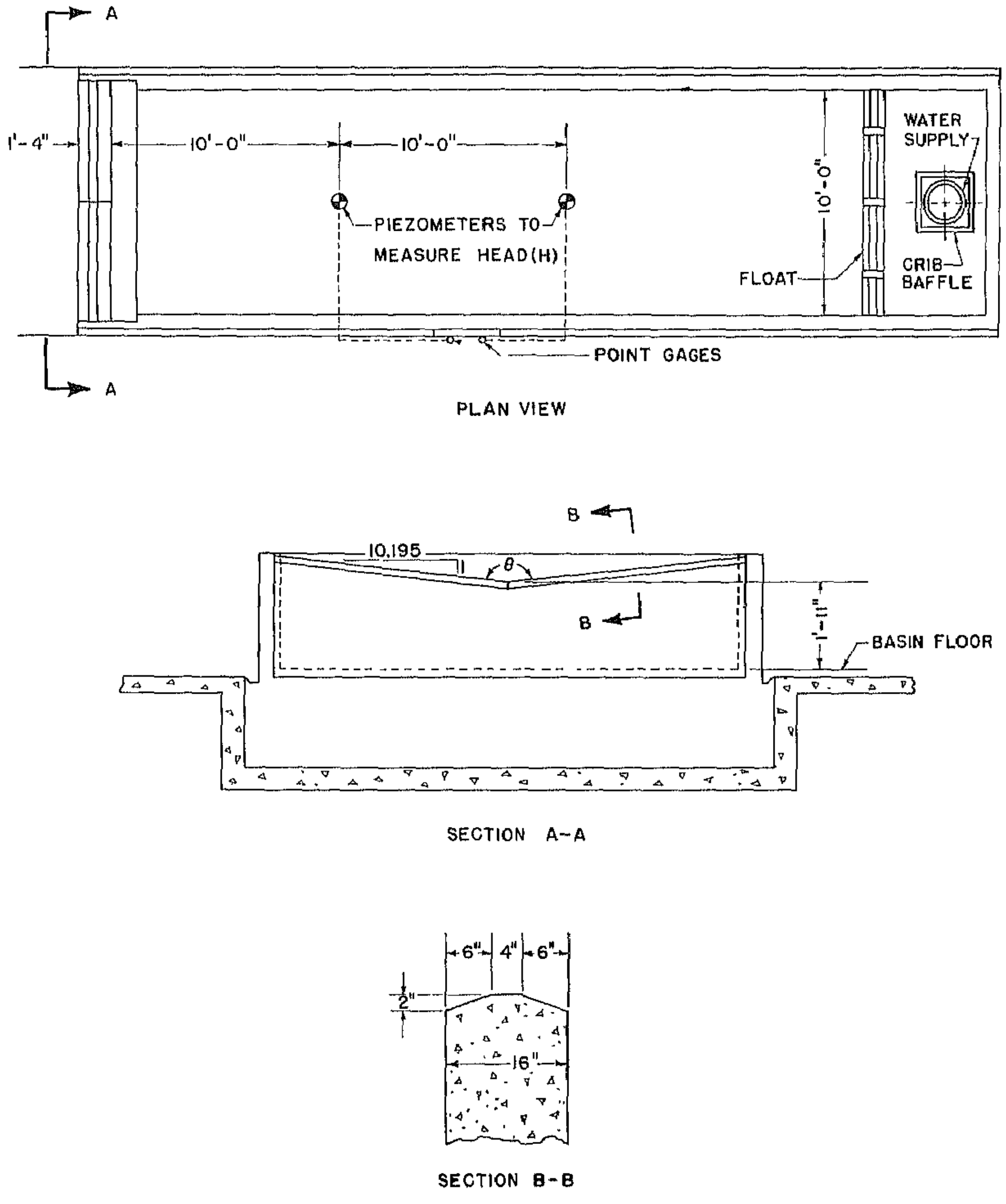


Figure 28.—Full-scale model of 16-inch broad-crested V-notch weir in laboratory basin.

The two empirical curves shown in figure 32 were obtained by applying the method of least squares to the transforms $Y = \log_{10}(1 - Q/Q_F)$ and $X = \log_{10}(e^{T/H})$. The following equation was obtained for the 8.005-ft weir length and discharges of 2.0, 1.0, and 0.5 ft³/s:

$$Q/Q_F = 1 - 7.464 \times 10^{-7} (e^{T/H})^{11}, \qquad (6)$$

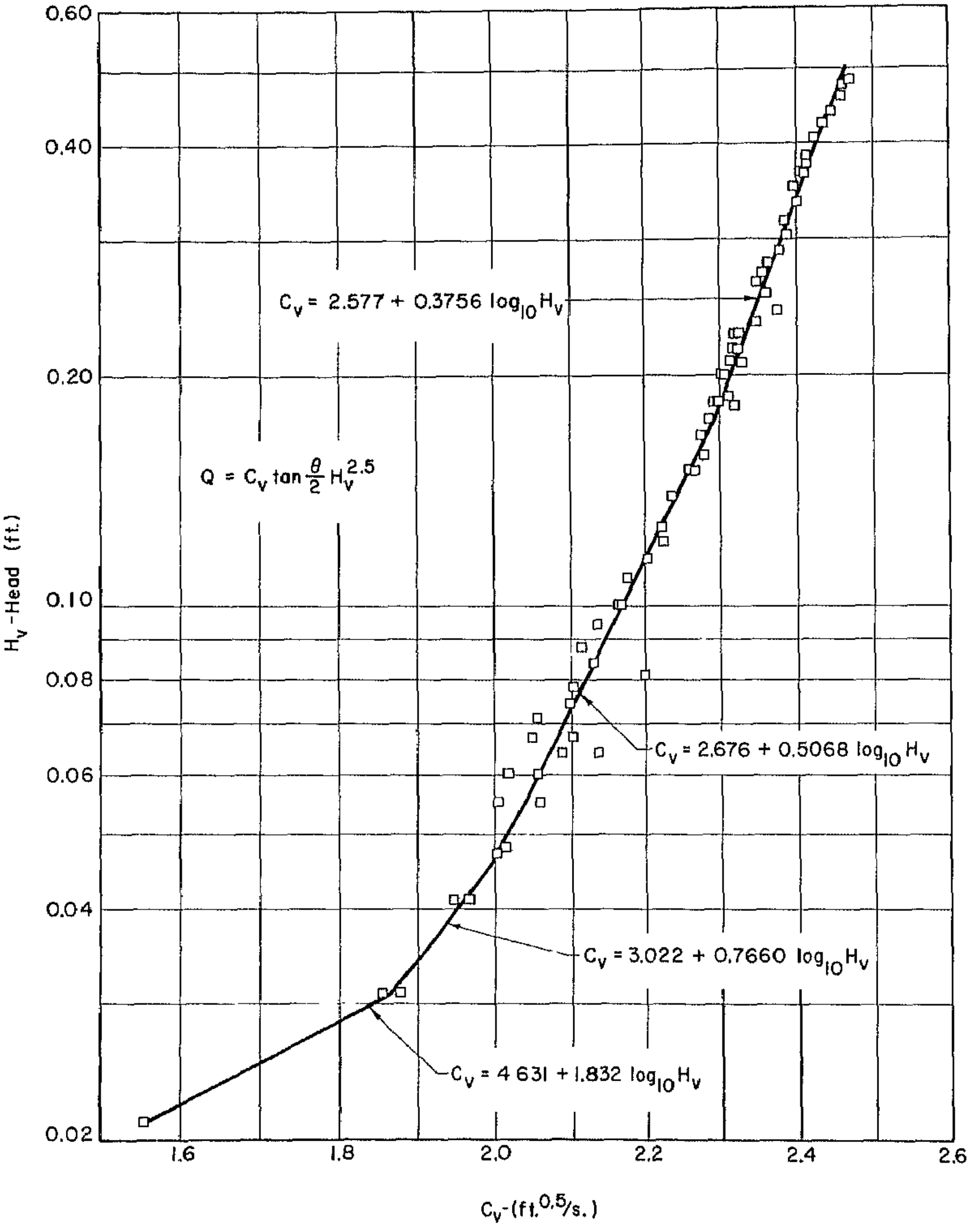


Figure 29.—Discharge coefficients for 16-inch broad-crested 1-in-10 V-notch weir.

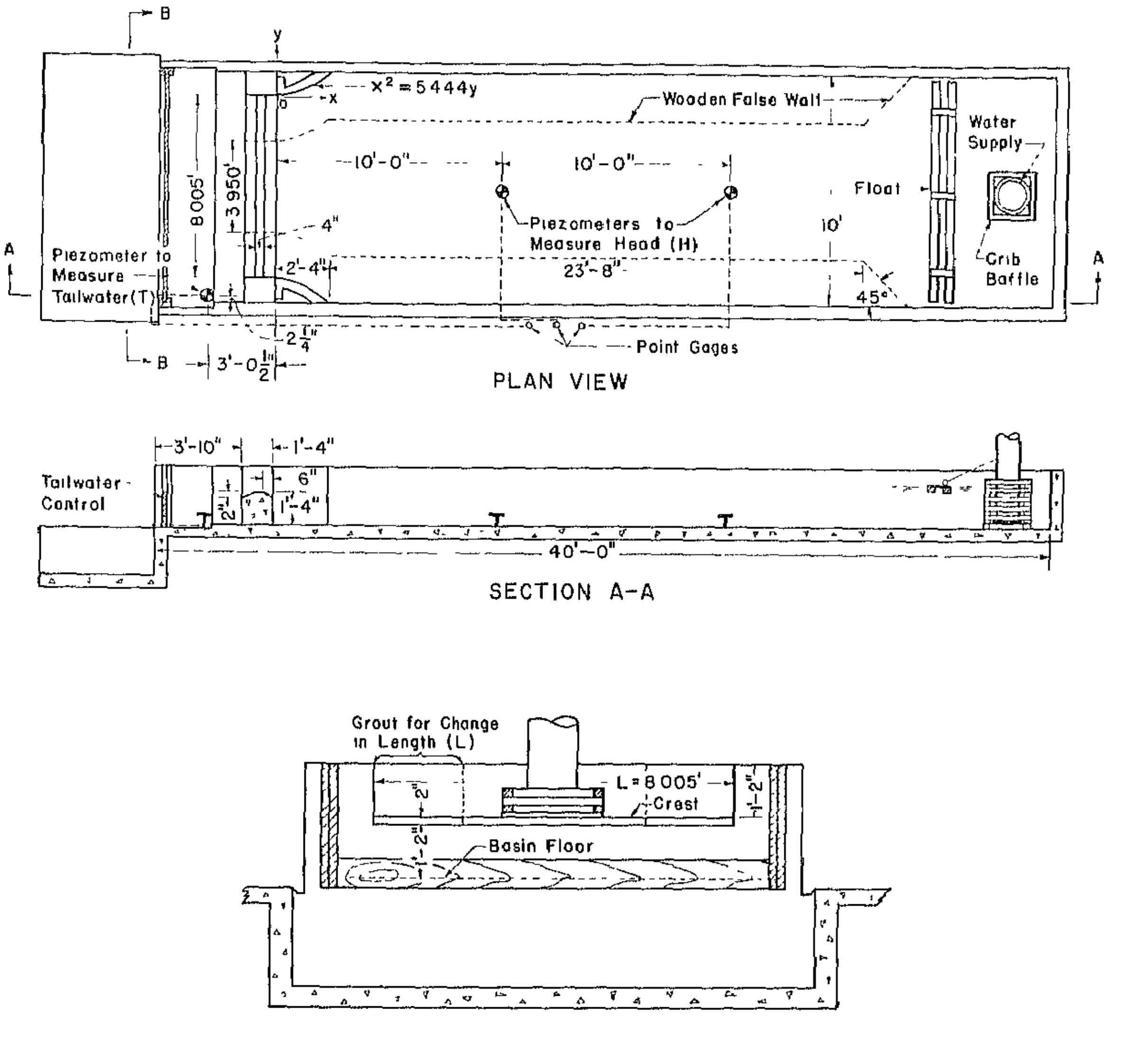


FIGURE 30.—Arrangement of the experiment for the low-flow calibration of horizontal, broad-crested weirs.

where Q = actual discharge (cubic feet per second) for submerged flow,
Q_F = free discharge (cubic feet per second) for equivalent head,
e = 2.7183 . . . ,
T = tailwater elevation above crest (feet),
and H = head (less than 0.29 ft).

The following equation was obtained for flows with head values of 0.29 ft and larger:

$$Q/Q_F=1-5.549\times10^{-6}(e^{T/H})^{12.13}, \qquad (7)$$

where $H > 0.29$ ft.

The equation of discharge for submerged flows over horizontal, 16-in. broad-crested weirs for upstream heads H equal to or greater than 0.29 ft can be obtained by substituting for Q_F in equation the equivalent for Q from equation 5, and the appropriate value for C_F from figure 31. The resulting equation is:

$$Q=(3.471+0.5675\ \log_{10}H)\,LH^{1.5}\,[1-5.549\times10^{-6}(e^{T/H})^{12.13}] \qquad (8)$$

Similar equations can be written for heads less than 0.29 ft by using the appropriate values for C_F from figure 31. The results of all tests are shown in table 4.

TABLE 3 —*Summary of data, 16-inch broad-crested 1-on-10 V-notch weir*

TEST NO.	DISCHARGE	10 FEET UPSTREAM H	10 FEET UPSTREAM C	20 FEET UPSTREAM H	20 FEET UPSTREAM C
1	2.263	0.385	2.41	0.385	2.41
2	3.975	0.479	2.46	0.479	2.46
3	4.072	0.483	2.47	0.483	2.47
4	3.617	0.461	2.46	0.461	2.46
5	3.209	0.441	2.44	0.440	2.45
6	2.938	0.426	2.43	0.426	2.43
7	2.627	0.408	2.42	0.408	2.42
8	2.153	0.377	2.41	0.377	2.41
9	2.006	0.367	2.41	0.367	2.41
10	1.796	0.352	2.40	0.352	2.40
11	1.601	0.336	2.40	0.336	2.40
12	1.387	0.318	2.39	0.318	2.39
13	1.240	0.304	2.39	0.304	2.39
14	1.097	0.290	2.38	0.290	2.38
15	0.999	0.280	2.36	0.280	2.36
16	0.927	0.272	2.36	0.272	2.36
17	0.858	0.264	2.35	0.264	2.35
18	0.790	0.255	2.36	0.255	2.36
19	0.705	0.243	2.38	0.243	2.38
20	0.641	0.235	2.35	0.235	2.35
21	0.580	0.227	2.32	0.227	2.32
22	0.519	0.217	2.32	0.217	2.32
23	0.470	0.209	2.31	0.208	2.33
24	0.420	0.200	2.31	0.200	2.30
25	0.344	0.185	2.30	0.185	2.30
26	0.361	0.188	2.31	0.188	2.31
27	0.338	0.183	2.32	0.183	2.32
28	0.299	0.175	2.29	0.175	2.29
29	0.259	0.166	2.28	0.166	2.28
30	0.226	0.157	2.28	0.157	2.28
31	0.200	0.150	2.26	0.150	2.27
32	0.164	0.139	2.24	0.139	2.24
33	0.128	0.126	2.22	0.126	2.22
34	0.115	0.121	2.22	0.121	2.22
35	0.101	0.115	2.20	0.115	2.20
36	0.085	0.108	2.18	0.108	2.18
37	0.070	0.100	2.17	0.100	2.17
38	0.059	0.094	2.14	0.094	2.14
39	0.050	0.088	2.11	0.088	2.11
40	0.044	0.084	2.13	0.084	2.13
41	0.041	0.081	2.20	0.081	2.20
42	0.036	0.078	2.11	0.078	2.11
43	0.032	0.074	2.10	0.074	2.10
44	0.028	0.071	2.06	0.071	2.06
45	0.025	0.067	2.07	0.067	2.07
46	0.022	0.064	2.13	0.064	2.13
47	0.018	0.060	2.04	0.060	2.04
48	0.015	0.055	2.03	0.055	2.03
49	0.010	0.048	2.01	0.048	2.01
50	0.007	0.041	1.96	0.041	1.96
51	0.003	0.031	1.87	0.031	1.87
52	0.001	0.021	1.56	0.021	1.56

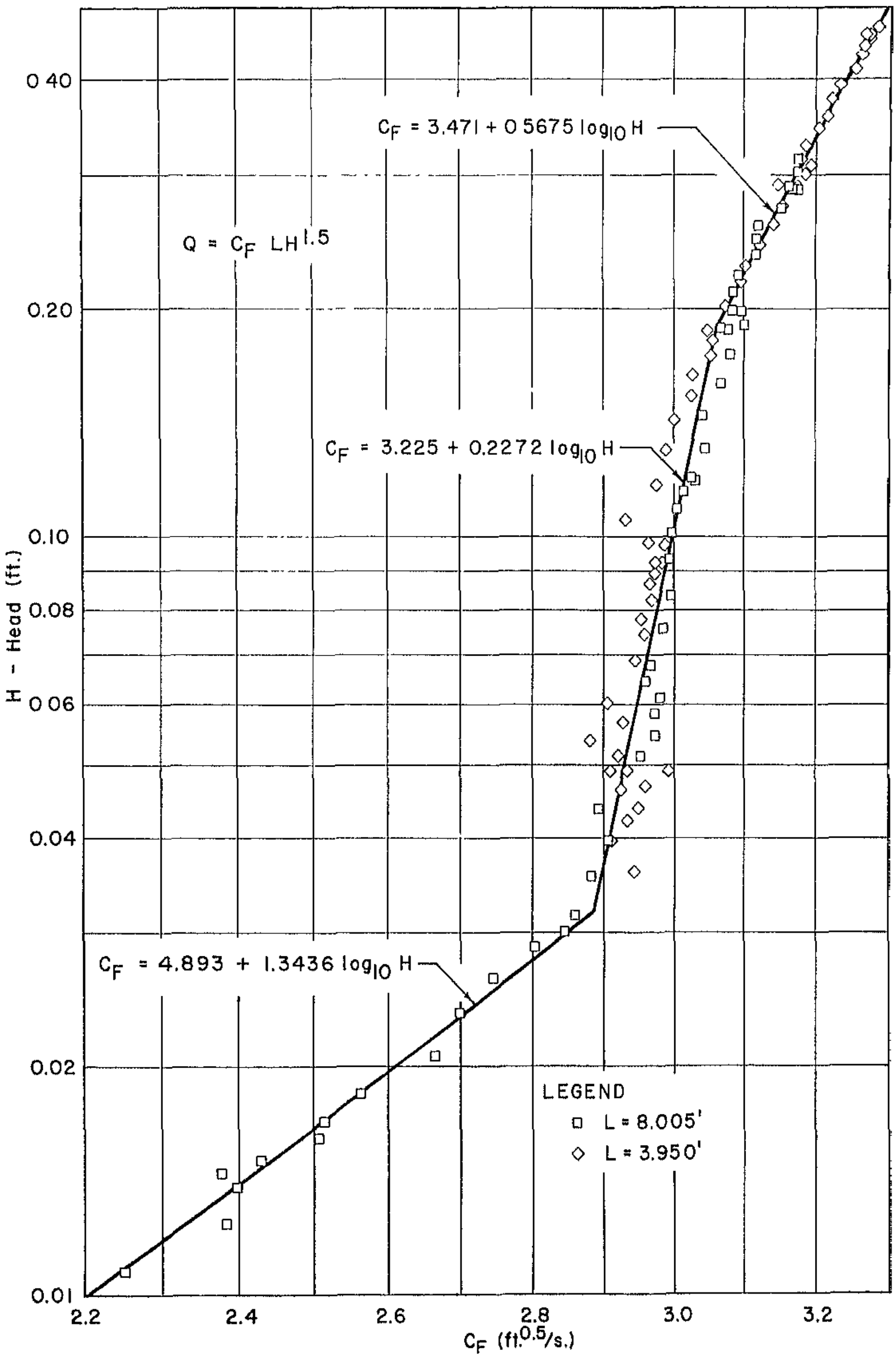


FIGURE 31.—Discharge coefficients (C_F) for horizontal, broad-crested weirs.

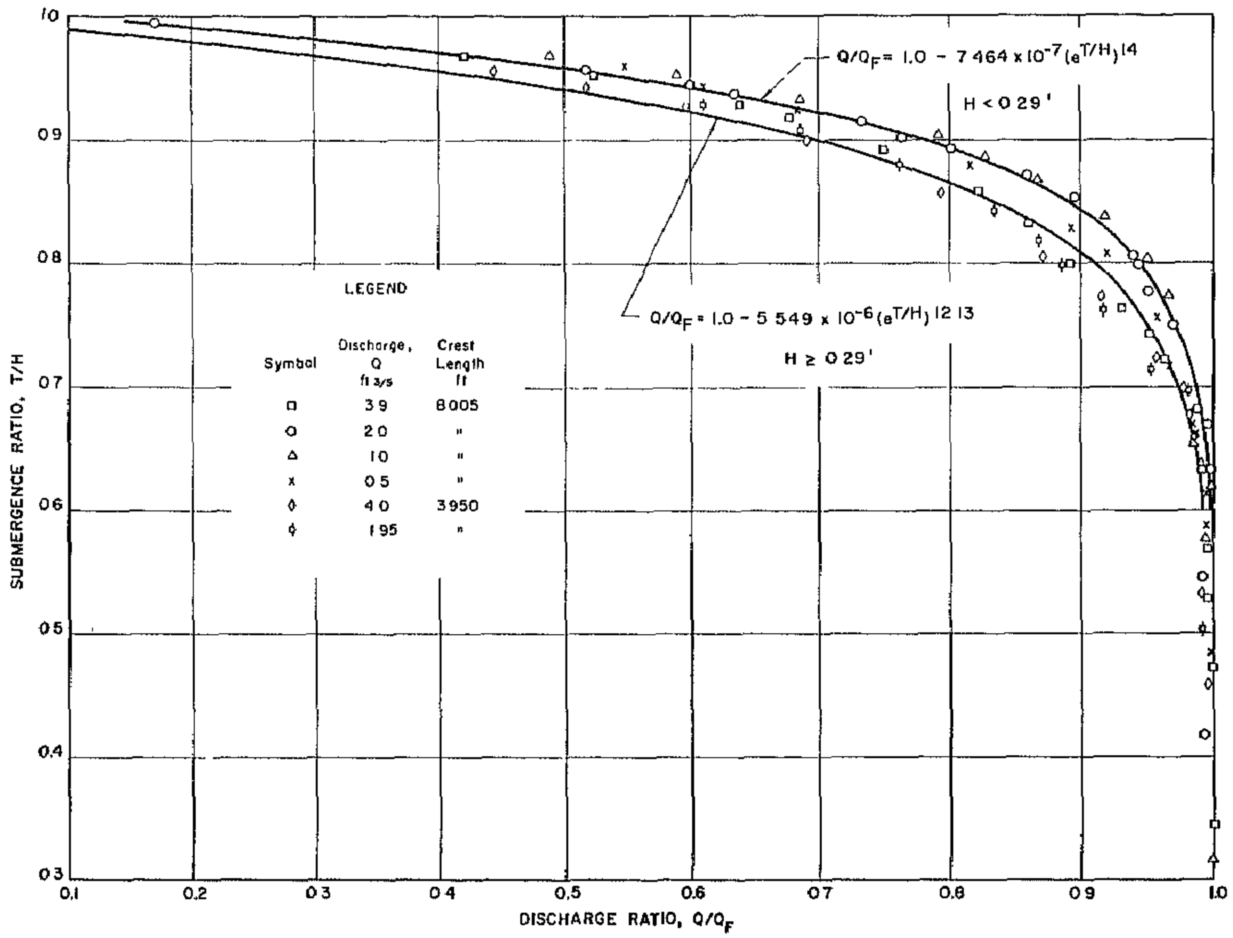


FIGURE 32.—Discharge-submergence relationship for horizontal broad-crested weirs.

TABLE 4.—*Summary of data, 16-inch horizontal, broad-crested weir*

TEST NO.	CREST LENGTH	DISCHARGE	TAILWATER T	10 FEET UPSTREAM H	C	20 FEET UPSTREAM H	C
1	8.005	2.054		0.190	3.10	0.189	3.11
2	8.005	1.793		0.174	3.08	0.174	3.09
3	8.005	1.551		0.159	3.07	0.158	3.08
4	8.005	1.336		0.144	3.04	0.144	3.06
5	8.005	1.150		0.131	3.04	0.130	3.06
6	8.005	0.989		0.118	3.03	0.118	3.04
7	8.005	2.193		0.199	3.09	0.198	3.11
8	8.005	3.908		0.287	3.18	0.286	3.19
9	8.005	3.573		0.272	3.15	0.271	3.16
10	8.005	3.247		0.257	3.12	0.256	3.13
11	8.005	2.856		0.236	3.12	0.235	3.13
12	8.005	2.564		0.221	3.09	0.220	3.10
13	8.005	2.376		0.210	3.08	0.210	3.09
14	8.005	2.196		0.199	3.08	0.199	3.09
15	8.005	3.056		0.247	3.12	0.246	3.13
16	8.005	3.963		0.290	3.17	0.289	3.18
17	8.005	0.936		0.115	3.01	0.114	3.03
18	8.005	0.861		0.109	3.00	0.108	3.02
19	8.005	0.774		0.101	3.00	0.101	3.01
20	8.005	0.681		0.093	2.99	0.093	2.99
21	8.005	0.578		0.083	2.99	0.083	3.00
22	8.005	0.495		0.075	2.99	0.075	2.99
23	8.005	0.418		0.068	2.97	0.068	2.97
24	8.005	0.385		0.064	2.96	0.064	2.96
25	8.005	0.361		0.061	2.98	0.061	2.98
26	8.005	0.334		0.058	2.97	0.058	2.97
27	8.005	0.304		0.055	2.97	0.055	2.97
28	8.005	0.273		0.051	2.95	0.051	2.95
29	8.005	0.239		0.047	2.94	0.047	2.94
30	8.005	0.211		0.044	2.89	0.044	2.90
31	8.005	0.184		0.040	2.91	0.040	2.90
32	8.005	0.155		0.036	2.88	0.036	2.88
33	8.005	0.129		0.032	2.86	0.032	2.86
34	8.005	0.118		0.030	2.85	0.030	2.83
35	8.005	0.109		0.029	2.80	0.029	2.80
36	8.005	0.093		0.026	2.75	0.026	2.75
37	8.005	0.078		0.024	2.70	0.023	2.72
38	8.005	0.063		0.021	2.67	0.021	2.67
39	8.005	0.051		0.018	2.56	0.018	2.58
40	8.005	0.044		0.017	2.52	0.017	2.52
41	8.005	0.041		0.016	2.51	0.016	2.51
42	8.005	0.033		0.014	2.38	0.014	2.38
43	8.005	0.026		0.012	2.39	0.012	2.42
44	8.005	0.020		0.011	2.25	0.011	2.28
45	8.005	0.014		0.009	2.10	0.009	2.14
46	8.005	0.036		0.015	2.43	0.015	2.43
47	8.005	0.031		0.014	2.40	0.014	2.40
48	8.005	4.479		0.314	3.18	0.313	3.20
49	8.005	4.230		0.302	3.18	0.302	3.18
50	8.005	3.943		0.289	3.16	0.289	3.17
51	8.005	2.001		0.188	3.07	0.187	3.08
51	8.005	1.999	0.273	0.286	1.64	0.285	1.64

TABLE 4.—*Summary of data, 16-inch horizontal, broad-crested weir*—Continued

TEST NO.	CREST LENGTH	DISCHARGE	TAILWATER T	10 FEET UPSTREAM H	10 FEET UPSTREAM C	20 FEET UPSTREAM H	20 FEET UPSTREAM C
51	8.004	2.003	0.569	0.572	0.58	0.572	0.58
51	8.005	2.003	0.103	0.189	3.05	0.188	3.06
51	8.005	2.002	0.078	0.188	3.06	0.188	3.06
51	8.005	2.003	0.048	0.188	3.06	0.188	3.07
51	8.005	1.996	0.128	0.189	3.04	0.189	3.04
51	8.005	2.002	0.150	0.194	2.94	0.194	2.93
51	8.005	2.000	0.172	0.201	2.77	0.201	2.76
51	8.005	2.001	0.193	0.216	2.48	0.216	2.48
51	8.005	2.001	0.209	0.229	2.28	0.229	2.28
51	8.005	2.002	0.246	0.260	1.89	0.260	1.89
51	8.005	2.001	0.235	0.251	1.99	0.251	1.99
51	8.005	2.000	0.201	0.223	2.37	0.223	2.37
51	8.005	1.999	0.180	0.207	2.66	0.206	2.66
51	8.005	1.997	0.157	0.195	2.90	0.195	2.90
51	8.005	2.005	0.126	0.188	3.07	0.188	3.07
51	8.005	1.995	0.118	0.187	3.07	0.187	3.08
51	8.005	2.004	0.156	0.195	2.91	0.195	2.91
51	8.005	2.003	0.145	0.193	2.96	0.193	2.96
51	8.005	2.001		0.188	3.08	0.187	3.09
52	8.005	1.002		0.120	3.03	0.120	3.03
52	8.005	1.001	0.038	0.120	3.02	0.120	3.02
52	8.005	1.002	0.074	0.120	3.02	0.120	3.02
52	8.005	0.997	0.069	0.120	3.01	0.120	3.01
52	8.005	1.005	0.087	0.121	2.97	0.121	2.97
52	8.005	1.002	0.079	0.121	2.98	0.121	2.98
52	8.005	1.001	0.095	0.122	2.93	0.122	2.93
52	8.005	1.002	0.099	0.124	2.88	0.124	2.88
52	8.005	1.002	0.106	0.126	2.79	0.126	2.79
52	8.005	1.002	0.114	0.131	2.64	0.131	2.64
52	8.005	1.001	0.120	0.135	2.52	0.135	2.52
52	8.005	1.001	0.126	0.139	2.41	0.139	2.41
52	8.005	1.001	0.142	0.153	2.10	0.153	2.10
52	8.005	1.002	0.161	0.169	1.81	0.169	1.81
52	8.005	1.000	0.184	0.190	1.51	0.190	1.51
53	8.005	0.499		0.076	2.98	0.076	2.99
53	8.005	0.498	0.037	0.076	2.98	0.076	2.98
53	8.005	0.499	0.045	0.076	2.97	0.076	2.97
53	8.005	0.499	0.047	0.076	2.97	0.076	2.97
53	8.005	0.500	0.051	0.077	2.95	0.077	2.95
53	8.005	0.499	0.056	0.078	2.88	0.078	2.89
53	8.005	0.499	0.051	0.077	2.94	0.077	2.94
53	8.005	0.499	0.059	0.078	2.86	0.078	2.86
53	8.005	0.500	0.065	0.080	2.75	0.080	2.75
53	8.005	0.498	0.068	0.082	2.67	0.082	2.67
53	8.005	0.498	0.077	0.087	2.42	0.087	2.42
53	8.005	0.501	0.090	0.098	2.05	0.098	2.05
53	8.005	0.499	0.099	0.105	1.84	0.105	1.84
53	8.005	0.499	0.108	0.112	1.66	0.112	1.66
53	8.005	0.498	0.117	0.121	1.49	0.121	1.49
54	8.005	3.901		0.287	3.17	0.287	3.17
54	8.005	3.900	0.182	0.289	3.14	0.289	3.14
54	8.005	3.901	0.163	0.288	3.15	0.288	3.15
54	8.005	3.895	0.152	0.288	3.15	0.288	3.15

TABLE 4.—*Summary of data, 16-inch horizontal, broad-crested weir*—Continued

TEST NO.	CREST LENGTH	DISCHARGE	TAILWATER T	10 FEET UPSTREAM H	C	20 FEET UPSTREAM H	C
54	8.005	3.893	0.197	0.290	3.12	0.289	3.12
54	8.005	3.898	0.229	0.301	2.96	0.301	2.96
54	8.005	3.902	0.246	0.309	2.84	0.309	2.84
54	8.005	3.906	0.262	0.316	2.75	0.316	2.75
54	8.005	3.909	0.279	0.325	2.63	0.325	2.63
54	8.005	3.903	0.307	0.344	2.41	0.344	2.41
54	8.005	3.902	0.337	0.368	2.19	0.367	2.19
54	8.005	3.904	0.353	0.381	2.07	0.381	2.08
54	8.005	3.898	0.411	0.432	1.71	0.432	1.72
54	8.005	3.903	0.481	0.497	1.39	0.496	1.39
54	8.005	3.900	0.220	0.296	3.02	0.296	3.03
54	8.005	3.904	0.212	0.294	3.05	0.294	3.07
54	8.005	3.901	0.135	0.287	3.16	0.286	3.18
54	8.005	3.901	0.099	0.287	3.17	0.286	3.19
54	8.005	3.902	0.047	0.287	3.17	0.286	3.19
55	3.950	4.170		0.469	3.29	0.468	3.30
56	3.950	3.962		0.454	3.28	0.453	3.29
57	3.950	3.723		0.437	3.27	0.436	3.27
58	3.950	3.646		0.431	3.26	0.430	3.27
59	3.950	3.414		0.413	3.25	0.413	3.26
60	3.950	3.164		0.394	3.24	0.394	3.24
61	3.950	2.951		0.377	3.23	0.377	3.23
62	3.950	2.714		0.357	3.22	0.357	3.22
63	3.950	2.568		0.345	3.21	0.345	3.21
64	3.950	2.357		0.327	3.19	0.327	3.20
65	3.950	2.213		0.314	3.18	0.314	3.19
66	3.950	2.168		0.309	3.20	0.308	3.20
67	3.950	2.077		0.301	3.19	0.300	3.20
68	3.950	1.951		0.289	3.18	0.289	3.18
69	3.950	1.788		0.274	3.15	0.274	3.16
70	3.950	1.627		0.258	3.14	0.258	3.15
71	3.950	1.472		0.242	3.12	0.242	3.13
72	3.950	1.330		0.228	3.10	0.227	3.11
73	3.950	1.234		0.217	3.09	0.216	3.10
74	3.950	1.098		0.202	3.07	0.201	3.08
75	3.950	0.974		0.187	3.05	0.187	3.06
76	3.950	0.934		0.182	3.06	0.181	3.07
77	3.950	0.869		0.173	3.05	0.173	3.06
78	3.950	0.792		0.164	3.03	0.163	3.04
79	3.950	0.716		0.153	3.02	0.153	3.04
80	3.950	0.639		0.143	3.00	0.142	3.02
81	3.950	0.554		0.130	2.99	0.130	3.00
82	3.950	0.469		0.117	2.98	0.116	3.00
83	3.950	0.395		0.105	2.93	0.105	2.96
84	3.950	0.360		0.098	2.96	0.098	2.99
85	3.950	0.332		0.093	2.97	0.093	2.98
86	3.950	0.298		0.086	2.97	0.086	2.98
87	3.949	0.253		0.078	2.95	0.078	2.96
88	3.949	0.210		0.069	2.94	0.069	2.96
89	3.949	0.169		0.060	2.90	0.060	2.93
90	3.949	0.142		0.054	2.88	0.054	2.89
91	3.949	0.129		0.049	2.99	0.049	2.99
92	3.949	0.119		0.047	2.96	0.047	2.97
93	3.949	0.107		0.044	2.95	0.044	2.95

TABLE 4.—*Summary of data, 16-inch horizontal, broad-crested weir*—Continued

TEST NO.	CREST LENGTH	DISCHARGE	TAILWATER T	10 FEET UPSTREAM H	C	20 FEET UPSTREAM H	C
94	3.949	0.091		0.040	2.91	0.040	2.91
95	3.949	0.080		0.036	2.94	0.036	2.94
96	3.949	0.068		0.033	2.91	0.033	2.93
97	3.949	0.057		0.029	2.92	0.029	2.92
98	3.949	0.047		0.026	2.84	0.026	2.84
99	3.949	0.100		0.042	2.93	0.042	2.93
100	3.949	0.125		0.049	2.91	0.049	2.92
101	3.949	0.044		0.025	2.83	0.025	2.83
102	3.949	0.041		0.024	2.84	0.024	2.86
103	3.949	0.036		0.022	2.81	0.022	2.81
104	3.949	0.031		0.020	2.72	0.020	2.74
105	3.949	0.023		0.017	2.66	0.017	2.66
106	3.949	0.015		0.013	2.49	0.013	2.52
107	3.950	3.995		0.457	3.28	0.456	3.28
107	3.950	4.004	0.209	0.459	3.26	0.458	3.27
107	3.950	4.004	0.312	0.463	3.22	0.462	3.23
107	3.950	3.998	0.323	0.463	3.21	0.463	3.22
107	3.950	3.993	0.292	0.459	3.25	0.459	3.25
107	3.950	4.008	0.340	0.471	3.14	0.470	3.15
107	3.950	4.007	0.373	0.484	3.01	0.483	3.02
107	3.950	4.007	0.401	0.499	2.88	0.498	2.88
107	3.950	4.002	0.453	0.529	2.63	0.528	2.64
107	3.950	3.997	0.520	0.578	2.30	0.577	2.31
107	3.950	3.992	0.587	0.633	2.01	0.633	2.01
107	3.950	4.015	0.656	0.696	1.75	0.695	1.75
107	3.950	3.996	0.730	0.764	1.51	0.763	1.52
107	3.950	4.001	0.244	0.460	3.25	0.459	3.25
107	3.950	4.003		0.458	3.27	0.457	3.28
108	3.950	1.948		0.291	3.15	0.290	3.16
108	3.950	1.946	0.146	0.291	3.14	0.291	3.14
108	3.950	1.950	0.192	0.292	3.12	0.292	3.13
108	3.950	1.954	0.213	0.299	3.02	0.299	3.03
108	3.950	1.948	0.204	0.293	3.11	0.293	3.12
108	3.950	1.948	0.233	0.306	2.92	0.305	2.92
108	3.950	1.951	0.250	0.313	2.82	0.312	2.83
108	3.950	1.951	0.259	0.317	2.77	0.316	2.77
108	3.950	1.951	0.273	0.325	2.66	0.325	2.67
108	3.950	1.949	0.302	0.344	2.45	0.343	2.45
108	3.950	1.949	0.333	0.368	2.21	0.367	2.22
108	3.950	1.947	0.367	0.396	1.98	0.395	1.98
109	3.950	0.359		0.098	2.99	0.097	3.01
110	3.950	0.332		0.093	2.98	0.092	3.00
111	3.950	0.316		0.090	2.97	0.089	3.00
112	3.949	0.276		0.082	2.97	0.082	3.00
113	3.949	0.236		0.074	2.96	0.074	2.99
114	3.949	0.194		0.065	2.96	0.065	3.00
115	3.949	0.156		0.057	2.93	0.056	2.97
116	3.949	0.134		0.051	2.92	0.051	2.96
117	3.949	0.126		0.049	2.93	0.049	2.96
118	3.949	0.115		0.046	2.92	0.046	2.97
119	3.949	0.084		0.038	2.87	0.038	2.93
120	3.949	0.077		0.036	2.84	0.036	2.90
121	3.949	0.066		0.033	2.79	0.032	2.85
122	3.949	0.054		0.029	2.75	0.029	2.82

APPENDIX C
Summary of 1:30 Model Data

TEST NO.	Q	HEADWATER ELEVATION GAGE 1	GAGE 2	GAGE 3	GAGE 4	GAGE 5	TAILWATER ELEVATION GAGE 6	GAGE 7	GAGE 8	V-NOTCH HEAD AV-GAGE 1 AND 2	GAGE 4	SCOUR TIME
1	17938	98.83	98.67	98.81	97.35	97.62				10.76	9.35	
2	19348	99.36	99.15	99.28	97.68	97.96	90.86	90.09	90.26	11.26	9.68	
3	16622	98.40	98.23	98.34	97.03	97.27				10.32	9.03	
4	15331	97.94	97.79	97.89	96.70	96.93				9.87	8.70	
5	13709	97.35	97.21	97.30	96.27	96.45				9.28	8.28	
6	12048	96.71	96.61	96.67	95.79	95.94				8.66	7.79	
7	4008	93.62	93.56	93.59	93.37	93.46	92.59	92.59	92.59	5.60	5.38	110
8	4010	93.48	93.42	93.45	93.18	93.33	91.43	91.41	91.40	5.45	5.18	110
9	4006	93.40	93.36	93.39		93.26	89.86	89.82	89.81	5.38		115
10	4001	98.63	98.60	98.60	98.51	98.55	98.49	98.49	98.48	10.62	10.51	
11	4007	96.88	96.85	96.85	96.75	96.78	96.69	96.69	96.70	8.87	8.76	
12	4006	96.19	96.16	96.16	96.04	96.08	95.95	95.95	95.95	8.18	8.05	
13	4008	94.98	94.94	94.95	94.81	94.84	94.62	94.62	94.62	6.96	6.82	
14	4015	94.42	94.38	94.39	94.21	94.27	93.94	93.94	93.93	6.40	6.22	
15	4002	93.83	93.79	93.79	93.60	93.67	93.07	93.07	93.07	5.81	5.60	
16	4001	93.52	93.48	93.49	93.24	93.39	92.32	92.32	92.31	5.51	5.25	
17	4001	93.43	93.39	93.40	93.13	93.31	91.74	91.72	91.71	5.41	5.14	
18	3997	93.37	93.34	93.37		93.25	91.08	91.05	91.03	5.36		
19	4000	93.34	93.30	93.34		93.23	90.28	90.24	90.22	5.33		
20	3995	93.34	93.28	93.31		93.22				5.32	4.99	
21	4327	93.45	93.43	93.46		93.35				5.45		
22	4550	93.58	93.52	93.55		93.43				5.56		
23	3548	93.13	93.10	93.13		93.06				5.12		
24	3091	92.92	92.89	92.91		92.87				4.91		
25	2758	92.74	92.71	92.74		92.71				4.73		
26	2399	92.56	92.53	92.56		92.54				4.55		
27	1994	92.35	92.32	92.35		92.34				4.34		
28	8015	95.42	95.32	95.35		94.96	93.76	93.79	93.79	7.38		120
29	8010	95.35	95.26	95.31		94.87	93.09	93.09	93.07	7.31		110
30	8010	95.82	95.73	95.74	95.37	95.38	94.51	94.56	94.55	7.78	7.37	115
31	8001	98.29	98.23	98.25	98.02	98.02	97.72	97.77	97.81	10.26	10.03	
32	8001	97.21	97.15	97.17	96.91	96.91	96.48	96.52	96.58	9.19	8.92	
33	8010	96.75	96.69	96.72	96.42	96.42	95.88	95.94	95.98	8.73	8.42	
34	8006	96.25	96.18	96.21	95.86	95.86	95.17	95.22	95.26	8.22	7.87	
35	8010	95.73	95.65	95.70	95.29	95.29	94.35	94.41	94.41	7.70	7.29	
36	8010	95.43	95.35	95.38	94.90	94.96	93.64	93.67	93.67	7.39	6.91	
37	8015	95.23	95.17	95.20	94.69	94.81	92.78	92.79	92.78	7.20	6.70	
38	8010	95.16	95.08	95.13	94.57	94.72	92.21	92.20	92.20	7.13	6.58	
39	8010	95.11	95.05	95.08	94.51	94.69	91.53	91.45	91.56	7.09	6.52	
40	8015	95.10	95.02	95.05	94.50	94.68	90.89	90.79	90.89	7.07	6.50	
41	8020	95.09	95.02	95.05	94.48	94.68	90.19	90.02	90.04	7.06	6.49	
42	10722	96.16	96.07	96.12	95.35	95.49				8.12	7.36	
43	9938	95.86	95.75	95.80	95.11	95.26				7.81	7.12	
44	8972	95.46	95.38	95.43	94.80	94.98				7.43	6.80	
45	7089	94.72	94.66	94.69	94.20	94.41				6.70	6.20	
46	6251	94.39	94.33	94.37	93.93	94.14				6.37	5.93	
47	5422	94.06	94.00	94.03	93.64	93.85				6.04	5.64	
48	1798	92.34	92.31	92.32	92.19	92.32				4.33	4.[illegible]	
49	1656	92.26	92.23	92.24	92.11	92.25				4.25	4.[illegible]	
50	1507	92.17	92.14	92.15	92.03	92.16				4.16	4[illegible]	

TEST NO.	Q	HEADWATER ELEVATION GAGE 1	GAGE 2	GAGE 3	GAGE 4	GAGE 5	TAILWATER ELEVATION GAGE 6	GAGE 7	GAGE 8	V-NOTCH HEAD AV-GAGE 1 AND 2	GAGE 4	SCOUR TIME
51	1333	92.05	92.02	92.04	91.93	92.05				4.04	3.94	
52	1130	91.96	91.93	91.95	91.84	91.95				3.95	3.85	
53	976	91.72	91.71	91.72	91.63	91.72				3.72	3.64	
54	752	91.58	91.57	91.57	91.51	91.57				3.58	3.51	
55	1003	92.05	92.02	92.02	91.99	92.05	91.55	91.54	91.56	4.04	4.00	
56	1005	91.84	91.84	91.84	91.76	91.84	90.68	90.67	90.67	3.85	3.79	115
57	997	91.78	91.78		91.72		89.97	89.95		3.79	3.73	110
58	1000	91.78	91.77		91.66			88.34		3.78	3.67	110
59	1001	92.72	92.71	92.70	92.68	92.70	92.61	92.59	92.59	4.72	4.68	
60	1001	92.28	92.26	92.26	92.23	92.26	92.07	92.05	92.06	4.27	4.24	
61	1003	92.00	91.99	91.99	91.94	91.99	91.42	91.42	91.42	4.00	3.94	
62	1002	91.93	91.92	91.93	91.88	91.93	90.97	90.96	90.97	3.93	3.88	
63	1002	91.88	91.88	91.87	91.81	91.88	90.43	90.40	90.43	3.88	3.82	
64	1002	91.86	91.84	91.84	91.77		90.13	90.13		3.85	3.77	
65	1003	91.81	91.78	91.79	91.72					3.80	3.73	
66	644	91.45	91.44		91.39					3.45	3.40	
67	599	91.39	91.39		91.34					3.40	3.34	
68	530	91.30	91.28		91.24					3.30	3.25	
69	477	91.18	91.15		91.12					3.17	3.13	
70	395	90.94	90.94		90.91					2.95	2.92	
71	324	90.74	90.73		90.72					2.74	2.72	
72	256	90.52	90.49		90.49					2.51	2.50	
73	300	90.99	90.98		90.96		90.42	90.42		2.99	2.96	
74	299	90.84	90.84		90.82		90.05	90.04		2.84	2.83	
75	299	91.29	91.28		91.26		91.02	91.01		3.29	3.26	
76	300	91.59	91.57		91.56		91.50	91.50		3.58	3.56	
77	300	91.47	91.46		91.44		91.34	91.34		3.47	3.44	
78	300	91.17	91.16		91.14		90.75	90.75		3.17	3.14	
79	300	90.84	90.82		90.80		89.97	89.97		2.83	2.81	113
80	300	90.72	90.69		90.66		89.31	89.31		2.71	2.66	110
81	300	90.66	90.64		90.60			88.33		2.65	2.60	110
82	300	90.80	90.78		90.75		89.67	89.67		2.79	2.75	
83	300	90.72	90.72		90.69		89.32	89.32		2.72	2.69	
84	220	90.30	90.30		90.30					2.30	2.30	
85	203	90.24	90.24		90.24					2.24	2.24	
86	184	90.16	90.16		90.16					2.16	2.16	
87	163	90.07	90.07		90.07					2.08	2.08	
88	141	89.97	89.97		89.97					1.97	1.97	
89	122	89.87	89.87		89.87					1.88	1.88	
90	107	89.79	89.79		89.79					1.79	1.79	
91	89	89.67	89.67		89.67					1.67	1.67	
92	71	89.55	89.55		89.55					1.55	1.55	
93	150	90.81	90.81		90.80		90.65	90.65		2.81	2.81	
94	150	90.50	90.49		90.49		90.23	90.23		2.50	2.50	
95	150	90.34	90.34		90.34		89.91	89.91		2.34	2.34	
96	150	90.23	90.21		90.21		89.58	89.58		2.22	2.21	
97	150	90.15	90.15		90.15		89.27	89.27		2.15	2.15	
98	150	90.10	90.09		90.09			88.89		2.10	2.09	
99	151	90.09	90.09		90.09			88.53		2.09	2.09	
100	152	90.09	90.08		90.08					2.09	2.09	126